GEOTECHNICAL INVESTIGATION
PROPOSED COMMERCIAL/INDUSTRIAL DEVELOPMENT

4300 Shirley Avenue
El Monte, California
for
Goodman Birtcher

SoCalGeo
SOUTHERN
CALIFORNIA
GEOTECHNICAL
A California Corporation

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February 3, 2016

Goodman Birtcher
18201 Von Karman Avenue, Suite 1170
Irvine, California 92612

Attention: Mr. Alan Tuntland
SVP Entitlements & Construction

Project No.: **15G227-1**

Subject: **Geotechnical Investigation**
Proposed Commercial/Industrial Development
4300 Shirley Avenue
El Monte, California

Gentlemen:

In accordance with your request, we have conducted a geotechnical investigation and liquefaction evaluation at the subject site. We are pleased to present this report summarizing the conclusions and recommendations developed from our investigation.

We sincerely appreciate the opportunity to be of service on this project. We look forward to providing additional consulting services during the course of the project. If we may be of further assistance in any manner, please contact our office.

Respectfully Submitted,

SOUTHERN CALIFORNIA GEOTECHNICAL, INC.

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TABLE OF CONTENTS

1.0 EXECUTIVE SUMMARY

Presented below is a brief summary of the conclusions and recommendations of this investigation. Since this summary is not all inclusive, it should be read in complete context with the entire report.

Geotechnical Design Considerations

- Our site-specific liquefaction evaluation indicates that some of the on-site soils are subject to liquefaction during the design seismic event. The liquefaction analysis indicates total dynamic settlements ranging between 0.5 and 3.2± inches at the four 50-foot deep boring locations. The liquefaction-induced differential settlements within the building areas are expected to be on the order of 2.7± inches. Assuming that the differential settlements occur across a distance of 100± feet, maximum angular distortions of less than 0.0025± inches per inch would result.
- Provided that the liquefaction induced settlements are considered in the structural design, it is considered feasible to support the proposed structures on shallow foundations.
- The subject site is underlain by existing fill soils, extending to depths of 1½ to 8½± feet. These existing fill soils are not considered suitable in their present condition, to support the foundations and floor slabs of the new buildings. Additionally, the near surface native alluvium possesses variable strengths and densities and a moderate potential for consolidation settlement. Remedial grading will be necessary to remove the existing fill soils and a portion of the near surface alluvial soils and replace them as compacted structural fill. Additionally, demolition of the existing structures is expected to cause significant disturbance to the onsite soils. Any soils disturbed during demolition should also be removed during remedial grading.
- It was not feasible, at the time of this geotechnical investigation, to drill a sufficient quantity of borings within the areas of proposed Buildings 1, 2, 3, and 8 to adequately characterize the subsurface conditions within these proposed building areas. At the time of subsurface exploration, the exploratory boring and trench areas were constrained by the ongoing operations of the Safeway distribution center, and the locations of the existing buildings, which were not accessible to our field personnel and drilling equipment. We recommend that a supplemental geotechnical investigation be performed within these proposed building areas, subsequent to the closure and/or demolition of the existing distribution center, in order to more completely characterize the subsurface conditions and confirm the suitability of the design recommendations provided in this report.

Site Preparation

- Demolition of the existing structures and pavements will be necessary in order to facilitate the construction of the proposed development. Demolition should include all foundations, floor slabs, utilities and any other subsurface improvements that will not remain in place with the new development. Concrete and asphalt debris may be crushed to a maximum 2-inch particle size, mixed with the on-site soils, and reused as compacted structural fill. It may also be feasible to crush these materials for use as crushed miscellaneous base (CMB).
- Initial site preparation should also include stripping of any surficial vegetation. Vegetation including grass and weed growth, trees, and any organic soils should be properly disposed of

off-site. Root balls associated with the trees should be removed in their entirety, and the resultant excavations should be backfilled with compacted structural fill soils.

- Remedial grading is recommended to be performed within the proposed building areas in order to remove all of the artificial fill soils and the upper portion of the alluvial soils. The existing soils within the proposed building areas should be overexcavated to a depth of 5 feet below existing grades and to a depth of at least 5 feet below the proposed building pad subgrade elevations. The depth of overexcavation should also be sufficient to remove any existing fill soils and any soils disturbed during demolition.
- The proposed foundation influence zones should be overexcavated to a depth of 3 feet below proposed foundation bearing grade.
- Following evaluation of the subgrade by the geotechnical engineer, the exposed subgrade soils should be scarified, moisture conditioned to 2 to 4 percent above optimum, and recompacted. The resulting soils may be replaced as compacted structural fill.
- The new parking and drive area subgrade soils are recommended to be scarified to a depth of 12± inches, thoroughly moisture conditioned and recompacted.

Building Foundations

- Conventional shallow foundations, supported in newly placed compacted fill.
- 2,500 lbs/ft^2 maximum allowable soil bearing pressure.
- Reinforcement consisting of at least six (6) No. 5 rebars (3 top and 3 bottom) in strip footings due to the presence of potentially liquefiable soils. Additional reinforcement may be necessary for structural considerations.

Building Floor Slab

- Conventional Slab-on-Grade, 5 inches thick.
- Minimum reinforcement of the floor slab should consist of No. 3 bars at 18-inches on center in both directions, due to the presence of potentially liquefiable soils. The actual floor slab reinforcement should be determined by the structural engineer, based on the imposed slab loading.
- Modulus of Subgrade Reaction: 100 psi/in

Pavements

ASPHALT PAVEMENTS (R = 40)					
Materials	**Thickness (inches)**				
	Auto Parking (TI = 4.0)	Auto Drive Lanes (TI = 5.0)	Truck Traffic		
			(TI = 6.0)	(TI = 7.0)	(TI = 8.0)
Asphalt Concrete	3	3	3½	4	5
Aggregate Base	3	4	6	7	8
Compacted Subgrade	12	12	12	12	12

PORTLAND CEMENT CONCRETE PAVEMENTS				
Materials	**Thickness (inches)**			
	Auto Parking & Drives (TI = 5.0)	Truck Traffic		
		(TI = 6.0)	(TI = 7.0)	(TI = 8.0)
PCC	5	5½	6½	8
Compacted Subgrade (95% Relative Compaction)	12	12	12	12

2.0 SCOPE OF SERVICES

The scope of services performed for this project was in accordance with our Proposal No. 15P405R, dated November 30, 2015. The scope of services included a visual site reconnaissance, subsurface exploration, field and laboratory testing, and geotechnical engineering analysis to provide criteria for preparing the design of the building foundations, building floor slab, and parking lot pavements along with site preparation recommendations and construction considerations for the proposed development. Based on the location of the subject site, this investigation also included a site specific liquefaction evaluation. The evaluation of the environmental aspects of this site was beyond the scope of services for this geotechnical investigation.

3.0 SITE AND PROJECT DESCRIPTION

3.1 Site Conditions

The subject site is located at the southeast corner of Lower Azusa Road and Shirley Avenue in El Monte, California. The street address for the site is 3400 Shirley Avenue. The site is bounded to the north by Lower Azusa Road, to the east by Gidley Elementary School, two commercial buildings and Arden Drive, to the south by a railroad easement, and to the west by Shirley Avenue. The general location of the site is illustrated on the Site Location Map, included as Plate 1 in Appendix A of this report.

The overall site is an irregular-shaped parcel, 93.31± acres in size. The western portion of the site is presently developed with a distribution center consisting of several buildings occupied by Safeway. The existing distribution center occupies the majority of the site area. The buildings are one and two-story structures of concrete tilt-up construction, presumably supported on conventional shallow foundations with concrete slab-on-grade floors. These buildings range from approximately 17,000± ft^2 to 425,520± ft^2 in size and are part of an existing Safeway distribution center. A previously existing warehouse building was recently demolished in the northwestern area of the site. The ground surface cover in this area consists of crushed asphalt and crushed aggregate base. Two (2) pump houses are located in the western half of the overall site. One pump house is located north of the previously demolished warehouse building and another is located in the southeast corner of this site. An above ground storage tank is present in the southeast corner of the site, adjacent to the previously mentioned pump house. Two (2) fuel pumps and underground storage tanks (USTs) are present in the south central region of the existing distribution center. Ground surface cover at the site consists of asphaltic concrete pavements in the automobile parking and drive lane areas and Portland cement concrete (PCC) pavements in the loading dock areas. The pavements are in poor to fair condition with moderate cracking throughout and areas of severe cracking and fatigue.

The remainder of the subject site, located east of the existing distribution center site is presently developed with several unoccupied commercial/industrial buildings and canopy structures. The buildings are one and two-story structures of masonry block and brick construction, presumably supported on shallow foundations with concrete slab-on-grade floors. These buildings range from approximately 5,500± ft^2 to 245,000± ft^2 in size. The canopy structures, constructed with sheet metal and steel columns, range from approximately 900± ft^2 to 5,500± ft^2 in size and are located in the southeastern region of this site. Several above ground storage tanks are present in the southeastern corner of the site. The central region of the site is currently being utilized as a parking lot for storage of automobiles by an auto dealer. Ground surface cover throughout this portion of the site generally consists of asphaltic concrete and Portland cement concrete (PCC) pavements. The PCC pavements are present in the truck loading dock areas in the central region of the site. The pavements are in fair to poor condition with moderate to severe cracking throughout. Limited areas of landscape planters and PCC flatwork are located throughout the site.

Detailed topographic information including existing site grades was not available at the time of this report. Based on visual observations made at the time of the subsurface investigation, the overall site topography generally slopes downward to the southeast at an estimated gradient of less than 2± percent.

3.2 Proposed Development

Our office was provided with a conceptual site plan by HPA, identified as Scheme 5. Based on this site plan, ten (10) new commercial/industrial buildings, identified as Building 1 through Building 10, will be constructed throughout the site. The buildings will be roughly rectangular in shape and will range in size from 28,840± ft^2 to 568,960± ft^2. We expect that the buildings will be surrounded by asphaltic concrete pavements in automobile parking and drive lanes and Portland cement concrete pavements in the truck loading areas. The remaining areas are expected to be developed with landscaped planters and concrete flatwork.

Detailed structural information has not been provided. It is assumed that the buildings will be single-story structures of tilt-up concrete construction, typically supported on conventional shallow foundation systems with concrete slab-on-grade floors. Based on the assumed construction, maximum column and wall loads are expected to be on the order of 80 kips and 3 to 5 kips per linear foot, respectively.

The proposed development is not expected to include any significant amounts of below grade construction such as basements or crawl spaces. Based on our review of the preliminary site plan, fills of 4 to 6± feet will be required in order to achieve the proposed building pad grades.

4.0 SUBSURFACE EXPLORATION

4.1 Scope of Exploration/Sampling Methods

The subsurface exploration performed for this project consisted of nineteen (19) borings advanced to depths of 5 to 50± feet below existing site grades. Four (4) of the borings were drilled to a depth of 50± feet, as part of the liquefaction evaluation. In addition to the borings, a total of five (5) trenches were excavated at the site to depths of 2½ to 3± feet below existing site grades. The trenches were performed in the northwest portion of the site, in the location of a previously demolished building. The purpose of the exploratory trenches was to determine if subsurface remnants of the former building, including foundations, were present below the ground surface in the former building area. All of the borings and trenches were logged during drilling and excavation by a member of our staff.

The borings were advanced with hollow-stem augers, by a truck-mounted drilling rig. The trenches were excavated using a rubber tire backhoe with a 36-inch wide bucket. Representative bulk and in-situ soil samples were taken during drilling and excavation. Relatively undisturbed samples were taken with a split barrel "California Sampler" containing a series of one inch long, 2.416± inch diameter brass rings. This sampling method is described in ASTM Test Method D-3550. Relatively undisturbed samples were also taken using a 1.4± inch inside diameter split spoon sampler, in general accordance with ASTM D-1586. Both of these samplers are driven into the ground with successive blows of a 140-pound weight falling 30 inches. The blow counts obtained during driving are recorded for further analysis. Bulk samples were collected in plastic bags to retain their original moisture content. The relatively undisturbed ring samples were placed in molded plastic sleeves that were then sealed and transported to our laboratory.

After the completion of drilling at each boring location, the drill cuttings were sealed in 55-gallon drums, each of which were properly labeled indicating the contents, the date filled, and the appropriate contact information. The drums were placed in temporary storage at the site pending profiling. The borings were properly sealed from the bottom-up with neat cement grout. The grout was mixed and placed through the hollow stem augers as the augers were withdrawn from the borings. Grout levels in the borings were checked periodically and additional grout was added as necessary such that the grout seal extended to within 12± inches of the ground surface. The upper 12± inches were backfilled with native soil and cold asphaltic concrete patch.

The approximate locations of the borings are indicated on the Boring Location Plan, included as Plate 2 in Appendix A of this report. The Boring Logs, which illustrate the conditions encountered at the boring locations, as well as the results of some of the laboratory testing, are included in Appendix B.

4.2 Geotechnical Conditions

Pavements

Asphaltic concrete pavements were present at the ground surface at all of the boring locations with the exception of Boring Nos. B-9 and B-19, which were drilled in areas of exposed soil and open graded gravel, respectively. The pavement sections consist of 2 to 6± inches of asphaltic concrete with 0 to 5± inches of underlying aggregate base. However, at Boring No. B-5, the pavement section consists of 5½± inches of asphaltic concrete with 24± inches of underlying aggregate base.
The ground surface cover at all of the exploratory trench locations consisted of 6± inches of crushed asphaltic concrete. Trench Nos. T-2 and T-5 encountered an additional asphaltic concrete pavement buried beneath the crushed asphaltic concrete, overlain by approximately 1 foot of fill. This buried pavement section consisted of 4± inches of asphaltic concrete with no discernible underlying aggregate base layer.

Artificial Fill

Artificial fill soils were encountered beneath the pavement at most of the boring locations and all of the exploratory trench locations, extending to depths of 1½ to 8½± feet below the existing site grades. The fill soils generally consist of very loose to medium dense silty fine to medium sands, fine sandy silts, and fine to coarse sands. The fill soils possess a disturbed appearance and trace amounts of asphalt fragments, resulting in their classification as fill.

Additional soils classified as possible fill were encountered at several of the boring locations, either at the ground surface or directly beneath the artificial fill materials. The possible fill materials encountered at Boring Nos. B-1, B-3, B-9, B-11, B-12, B-16, and B-19 vary in composition and consist of silty fine sands to fine sandy silts and clayey fine sands to fine sandy clays. These soils extend to depths of 1½ to 8± feet, and do not resemble the native alluvium encountered at similar depths at other boring locations, but lack obvious indicators of fill, such as extensive disturbance or artificial debris. These materials should be evaluated by a representative of the geotechnical engineer at the time of grading to determine whether they consist of artificial fill or alluvium.

Alluvium

Native alluvium was encountered beneath the pavements or below the artificial fill soils at all of the boring locations extending to at least the maximum depth explored of 50± feet below existing site grades. The alluvium generally consists of loose to very dense fine sands, silty fine sands, fine sandy silts, and fine to coarse sands with varying amounts of fine to coarse gravel, clay and silt, extending to the maximum depth explored of 50± feet below existing site grades.

Groundwater

Free water was not encountered during the drilling of any of the borings nor during the excavation of any of the exploratory trenches. Based on the lack of any water within the borings, and the moisture contents of the recovered soil samples, the static groundwater is considered to have existed at a depth in excess of 50± feet at the time of the subsurface exploration.

As part of our research, we reviewed available groundwater data in order to determine the historic high groundwater level for the site. The primary reference used to determine the historic groundwater depths in this area is CGS Open File Report 98-15, the Seismic Hazard Evaluation of the El Monte Quadrangle which indicates that the historic high groundwater levels for the site ranged between 10 feet near the southern portion of the site to 20± feet in the northeast portion of the site. More recent ground water level data was obtained from the California Department of Water Resources website, http://www.water.ca.gov/waterdatalibrary/. The nearest monitoring well is located approximately 2.5 miles east from the site. Water level readings within this monitoring well indicates high groundwater levels of 144± feet (July 2015) below the ground surface. Therefore, the historic high groundwater depths of 10 to 20± feet reported in OFR 98-15 is considered to be very conservative with respect to the recent site conditions.

5.0 LABORATORY TESTING

The soil samples recovered from the subsurface exploration were returned to our laboratory for further testing to determine selected physical and engineering properties of the soils. The tests are briefly discussed below. It should be noted that the test results are specific to the actual samples tested, and variations could be expected at other locations and depths.

Classification

All recovered soil samples were classified using the Unified Soil Classification System (USCS), in accordance with ASTM D-2488. Field identifications were then supplemented with additional visual classifications and/or by laboratory testing. The USCS classifications are shown on the Boring and Trench Logs and are periodically referenced throughout this report.

Dry Density and Moisture Content

The density has been determined for selected relatively undisturbed ring samples. These densities were determined in general accordance with the method presented in ASTM D-2937. The results are recorded as dry unit weight in pounds per cubic foot. The moisture contents are determined in accordance with ASTM D-2216, and are expressed as a percentage of the dry weight. These test results are presented on the Boring Logs.

Consolidation

Selected soil samples have been tested to determine their consolidation potential, in accordance with ASTM D-2435. The testing apparatus is designed to accept either natural or remolded samples in a one-inch high ring, approximately 2.416 inches in diameter. Each sample is then loaded incrementally in a geometric progression and the resulting deflection is recorded at selected time intervals. Porous stones are in contact with the top and bottom of the sample to permit the addition or release of pore water. The samples are typically inundated with water at an intermediate load to determine their potential for collapse or heave. The results of the consolidation testing are plotted on Plates C-1 through C-19 in Appendix C of this report.

Grain Size Analysis

Limited grain size analyses have been performed on several selected samples, in accordance with ASTM D-1140. These samples were washed over a #200 sieve to determine the percentage of fine-grained material in each sample, which is defined as the material which passes the #200 sieve. The weight of the portion of the sample retained on each screen is recorded and the percentage finer or coarser of the total weight is calculated. The results of these laboratory tests are shown on the attached boring logs.

Atterberg Limits

Atterberg Limits testing (ASTM D-4318) was performed on selected samples of various soil strata encountered at the site. This test is used to determine the Liquid Limit and Plastic Limit of the soil. The Plasticity Index is the difference between the two limits. Plasticity Index is a general indicator of the expansive potential of the soil, with higher numbers indicating higher expansive potential. Soils with a PI greater than 25 are considered to have a high plasticity, and a high expansion potential. Soils with a PI greater than 18 are not considered to be susceptible to liquefaction when the moisture content of the soil is less than 80 percent of the liquid limit. The results of the Atterberg Limits testing are presented on the boring logs.

Soluble Sulfates

Representative samples of the near-surface soils were submitted to a subcontracted analytical laboratory for determination of soluble sulfate content. Soluble sulfates are naturally present in soils, and if the concentration is high enough, can result in degradation of concrete which comes into contact with these soils. The results of the soluble sulfate testing are presented below, and are discussed further in a subsequent section of this report.

Sample Identification	**Soluble Sulfates (%)**	**ACI Classification**
B-1 @ 0 to 5 feet	0.0022	Negligible
B-4 @ 0 to 5 feet	0.0018	Negligible
B-8 @ 0 to 5 feet	0.0046	Negligible
B-10 @ 0 to 5 feet	0.0105	Negligible
B-15 @ 0 to 5 feet	0.0021	Negligible

Expansion Index

The expansion potential of the on-site soils was determined in general accordance with ASTM D-4829 as required by the California Building Code. The testing apparatus is designed to accept a 4-inch diameter, 1-in high, remolded sample. The sample is initially remolded to 50± 1 percent saturation and then loaded with a surcharge equivalent to 144 pounds per square foot. The sample is then inundated with water, and allowed to swell against the surcharge. The resultant swell or consolidation is recorded after a 24-hour period. The results of the EI testing are as follows:

Sample Identification	**Expansion Index**	**Expansive Potential**
B-3 @ 0 to 5 feet	58	Medium
B-15 @ 0 to 5 feet	15	Very Low

Maximum Dry Density and Optimum Moisture Content

Representative bulk samples were tested for their maximum dry densities and optimum moisture contents. The results have been obtained using the Modified Proctor procedure, per ASTM D-1557. These tests are generally used to compare the in-situ densities of undisturbed field samples, and for later compaction testing. Additional testing of other soil type or soil mixes may be necessary at a later date. The results of the testing are plotted on Plates C-20 and C-22 in Appendix C of this report.

6.0 CONCLUSIONS AND RECOMMENDATIONS

Based on the results of our review, field exploration, laboratory testing and geotechnical analysis, the proposed development is considered feasible from a geotechnical standpoint. The recommendations contained in this report should be taken into the design, construction, and grading considerations. The recommendations are contingent upon all grading and foundation construction activities being monitored by the geotechnical engineer of record. The Grading Guide Specifications, included as Appendix D, should be considered part of this report, and should be incorporated into the project specifications. The contractor and/or owner of the development should bring to the attention of the geotechnical engineer any conditions that differ from those stated in this report, or which may be detrimental for the development.

6.1 Seismic Design Considerations

The subject site is located in an area which is subject to strong ground motions due to earthquakes. The performance of a site specific seismic hazards analysis was beyond the scope of this investigation. However, numerous faults capable of producing significant ground motions are located near the subject site. Due to economic considerations, it is not generally considered reasonable to design a structure that is not susceptible to earthquake damage. Therefore, significant damage to structures may be unavoidable during large earthquakes. The proposed structure should, however, be designed to resist structural collapse and thereby provide reasonable protection from serious injury, catastrophic property damage and loss of life.

Faulting and Seismicity

Research of available maps indicates that the subject site is not located within an Alquist-Priolo Earthquake Fault Zone. Therefore, the possibility of significant fault rupture on the site is considered to be low.

Seismic Design Parameters

The 2013 California Building Code (CBC) was adopted by all municipalities within Southern California on January 1, 2014. The CBC provides procedures for earthquake resistant structural design that include considerations for on-site soil conditions, occupancy, and the configuration of the structure including the structural system and height. The seismic design parameters presented below are based on the soil profile and the proximity of known faults with respect to the subject site.

The 2013 CBC Seismic Design Parameters have been generated using U.S. Seismic Design Maps, a web-based software application developed by the United States Geological Survey. This software application, available at the USGS web site, calculates seismic design parameters in accordance with the 2013 CBC, utilizing a database of deterministic site accelerations at 0.01 degree intervals. The table below is a compilation of the data provided by the USGS application. A copy of the output generated from this program is included as Plate E-1 in Appendix E of this report. A copy of the Design Response Spectrum, as generated by the USGS application is also included in Appendix E. Based on this output, the following parameters may be utilized for the subject site:

2013 CBC SEISMIC DESIGN PARAMETERS

Parameter		Value
Mapped Spectral Acceleration at 0.2 sec Period	S_S	2.476
Mapped Spectral Acceleration at 1.0 sec Period	S_1	0.815
Site Class	---	F*
Site Modified Spectral Acceleration at 0.2 sec Period	S_{MS}	2.476
Site Modified Spectral Acceleration at 1.0 sec Period	S_{M1}	1.223
Design Spectral Acceleration at 0.2 sec Period	S_{DS}	1.651
Design Spectral Acceleration at 1.0 sec Period	S_{D1}	0.815

*The 2013 CBC requires that Site Class F be assigned to any profile containing soils vulnerable to potential failure or collapse under seismic loading, such as liquefiable soils. For Site Class F, the site coefficients are to be determined in accordance with Section 11.4.7 of ASCE 7-10. However, Section 20.3.1 of ASCE 7-10 indicates that for sites with structures having a fundamental period of vibration equal to or less than 0.5 seconds, the site coefficient factors (F_a and F_v) may be determined using the standard procedures. The seismic design parameters tabulated above were calculated using the site coefficient factors for Site Class D, assuming that the fundamental period of the structure is less than 0.5 seconds. However, the results of the liquefaction evaluation indicate that the subject site is underlain by potentially liquefiable soils. Therefore, if any of the proposed structures will have a fundamental period greater than 0.5 seconds, a site specific seismic hazards analysis would be required and additional subsurface exploration would be necessary.

Ground Motion Parameters

The site acceleration used for the liquefaction evaluation was determined using maximum considered earthquake ground motions, as required by the 2013 CBC. The peak ground acceleration (PGA) was determined in accordance with Section 11.8.3 of ASCE 7-10. The parameter PGA_M is the maximum considered earthquake geometric mean (MCE_G) PGA, multiplied by the appropriate site coefficient from Table 11.8-1 of ASCE 7-10. The web-based software application U.S. Seismic Design Maps (described in the previous section) was used to determine PGA_M, which is 0.899g for the subject site. A portion of the program output is included as Plate 2 of this report. An associated earthquake magnitude was obtained from the 2008 USGS Interactive Deaggregation application available on the USGS website. The deaggregated modal magnitude is 6.66, based on the peak ground acceleration and NEHRP soil classification D.

Liquefaction

The California Geological Survey (CGS) has not yet conducted detailed seismic hazards mapping in the area of the subject site. The general liquefaction susceptibility of the site was determined by research of the CGS Open File Report (OFR) 98-15, the Seismic Hazard Evaluation of the El Monte Quadrangle. OFR 98-15 indicates that the subject site is located within a mapped liquefaction hazard zone. Therefore, the scope of this geotechnical investigation was expanded to include a site-specific liquefaction evaluation.

Liquefaction is the loss of strength in generally cohesionless, saturated soils when the pore-water pressure induced in the soil by a seismic event becomes equal to or exceeds the overburden pressure. The primary factors which influence the potential for liquefaction include groundwater table elevation, soil type and plasticity characteristics, relative density of the soil, initial confining pressure, and intensity and duration of ground shaking. The depth within which the occurrence of liquefaction may impact surface improvements is generally identified as the upper 50 feet below the existing ground surface. Liquefaction potential is greater in saturated, loose, poorly graded fine sands with a mean (d_{50}) grain size in the range of 0.075 to 0.2 mm (Seed and Idriss, 1971). Non-sensitive clayey (cohesive) soils which possess a plasticity index of at least 18 (Bray and Sancio, 2006) are generally not considered to be susceptible to liquefaction, nor are those soils which are above the historic static groundwater table.

The liquefaction analysis was conducted in accordance with the requirements of Special Publication 117A (CDMG, 2008), and currently accepted practice (SCEC, 1997). The liquefaction potential of the subject site was evaluated using the empirical method developed by Boulanger and Idriss (Boulanger and Idriss, 2008). This method predicts the earthquake-induced liquefaction potential of the site based on a given design earthquake magnitude and peak ground acceleration at the subject site. This procedure essentially compares the cyclic resistance ratio (CRR) [the cyclic stress ratio required to induce liquefaction for a cohesionless soil stratum at a given depth] with the earthquake-induced cyclic stress ratio (CSR) at that depth from a specified design earthquake (defined by a peak ground surface acceleration and an associated earthquake moment magnitude). CRR is determined as a function of the corrected SPT N-value $(N_1)_{60\text{-}cs}$, adjusted for fines content. The factor of safety against liquefaction is defined as CRR/CSR. Based on Special Publication 117A, a factor of safety of at least 1.3 is required in order to demonstrate that a given soil stratum is non-liquefiable. Additionally, in accordance with Special Publication 117A, clayey soils which do not meet the criteria for liquefiable soils defined by Bray and Sancio (2006), loose soils with a plasticity index (PI) less than 12 and moisture content greater than 85% of the liquid limit, are considered to be insusceptible to liquefaction. Non-sensitive soils with a PI greater than 18 are also considered non-liquefiable.

As part of the liquefaction evaluation, Boring Nos. B-1, B-8, B-9 and B-17 were extended to depths of 50± feet. The liquefaction analysis procedure is tabulated on the spreadsheet forms included in Appendix F of this report, using the data obtained from these borings. The liquefaction potential of the site was analyzed utilizing a PGA_M of 0.899g for a magnitude 6.66 seismic event.

The historic high groundwater depth was obtained from CGS OFR 98-15, the Seismic Hazard Evaluation of the El Monte Quadrangle, which indicates that historic high groundwater depths for the subject range between depths of 10 to 20± feet for the southern and northwestern portions of the site, respectively. As noted in Section 4.2 of this report, the historic high groundwater depths are very conservative with respect to recent groundwater depths near the subject site.

If liquefiable soils are identified, the potential settlements that could occur as a result of liquefaction are determined using the equation for volumetric strain due to post-cyclic reconsolidation (Yoshimine et. al, 2006). This procedure uses an empirical relationship between the induced cyclic shear strain and the corrected N-value to determine the expected volumetric strain of saturated sands subjected to earthquake shaking. This analysis is also documented on the spreadsheets included in Appendix F.

Conclusions and Recommendations

The results of the liquefaction analysis have identified potentially liquefiable soils at the site. Several potentially liquefiable strata are located at various depths between 10 and 50± feet at all four of the 50± foot deep borings. Soils which are located above the historic groundwater table, or possess factors of safety in excess of 1.3 are considered non-liquefiable. Settlement analyses were conducted for each of the potentially liquefiable strata.

Based on the settlement analysis (also tabulated on the spreadsheets in Appendix F) total dynamic (liquefaction induced) settlements of 2.39, 3.18, 1.63, and 0.46± inches are expected at Boring Nos. B-1, B-8, and B-9 and B-17, respectively, during the design level earthquake with historic high groundwater conditions. Based on these total settlements, differential settlements of up to 2.7± inches should be expected to occur during a liquefaction-inducing seismic event. The estimated differential settlement could be assumed to occur across a distance of greater than 100 feet, indicating a maximum angular distortion of less than 0.0025 inches per inch. These settlements are considered to be within the structural tolerances of typical buildings supported on shallow foundation systems. However, it should be noted that minor to moderate repairs, including repair of damaged drywall and stucco, etc., could be required after the occurrence of liquefaction-induced settlements.

Provided that the liquefaction induced settlements are considered in the structural design, it is considered feasible to support the proposed structures on conventional shallow foundations. The use of shallow foundation systems, as described in this report, is typical for buildings of this type, where they are underlain by the extent of liquefiable soils encountered at this site. The post-liquefaction damage that could occur within the buildings proposed for this site will also be typical of similar buildings in the vicinity of this project. However, if the owner determines that this level of potential damage is not acceptable, other geotechnical and structural options are available, including the use of ground improvement, deep foundations or mat foundations.

6.2 Geotechnical Design Considerations

General

The near surface soils consist of artificial fill soils and native alluvium. Artificial fill soils extend to depths of 1½ to 8½± feet at most of the borings and all of the exploratory trench locations. The fill soils generally possess variable densities and based on the results of laboratory testing, the fill soils possess moderate to high potentials for settlement due to consolidation and collapse. Additionally, no reports documenting the placement and compaction of these materials were provided to our office. Based on the these conditions, the existing fill materials, in their present condition, are not considered to suitable for the support of the proposed structures at the subject site. The near-surface alluvium within the upper 5 to 6± feet is also highly variable in density and possesses minor to moderate potentials for consolidation settlement. Based on these conditions, remedial grading is considered warranted within the proposed building areas in order to remove the artificial fill in its entirety and replace the upper portion of variable strength alluvium as compacted structural fill.

Since no borings were performed inside the existing buildings, the presence and extent of any undocumented fill soils beneath the existing structures is presently unknown. Additionally, the demolition of these structures including foundations, utilities, underground storage tanks, and any other subsurface improvements is expected to cause significant disturbance to the near surface soils. Therefore, the recommended remedial grading should also remove any soils disturbed during demolition, prior to the placement of any new compacted fill materials.

It was not feasible, at the time of this geotechnical investigation, to drill a sufficient quantity of borings within the areas of proposed Buildings 1, 2, 3, and 8 to adequately characterize the subsurface conditions within these proposed building areas. At the time of subsurface exploration, the exploratory boring and trench areas were constrained by the ongoing operations of the Safeway distribution center, and the locations of the existing buildings, which were not accessible to our field personnel and drilling equipment. **We recommend that a supplemental geotechnical investigation be performed within these proposed building areas, subsequent to the closure and/or demolition of the existing distribution center, in order to more completely characterize the subsurface conditions and confirm the suitability of the design recommendations provided in this report.**

As discussed in Section 6.1 of this report, potentially liquefiable soils were identified at this site. The presence of the recommended layer of newly placed compacted structural fill above these liquefiable soils will help to reduce surface manifestations that could occur as a result of liquefaction. The foundation design recommendations presented in the subsequent sections of this report also contain recommendations to provide additional rigidity in order to reduce the potential effects of differential settlement that could occur as a result of liquefaction.

Settlement

The recommended remedial grading will remove the undocumented fill soils and a portion of the near surface, variable density alluvial soils from the proposed building pad areas. These materials will be replaced as compacted structural fill. The native soils that will remain in place below the recommended depth of overexcavation generally possess more favorable consolidation characteristics and will not be subject to significant load increases from the foundations of the new structure. Provided that the recommended remedial grading is completed, the post-construction static settlement of the proposed structure is expected to be within tolerable limits.

Expansion

The majority of near-surface soils encountered at the boring and trench locations consist of sands and silty sands. These materials have been visually classified as very low to non-expansive. Additionally the results of expansion index testing performed on a silty sand with little clay content at Boring No. B-15 indicated that this material soil possesses a very low expansion potential (EI=15). However, a few of the borings did encounter clayey fine sands and fine sandy clays within the upper 6± feet. An expansion index test performed on a sample obtained from the upper 5± feet at Boring on No. B-3 indicated that these soils possess a medium expansion index (EI = 58). Based on the assumed cuts and fills, and the recommended remedial grading, it is considered feasible to construct building pads with very low expansion potentials (EI < 20), either through blending the expansive clayey soils with the very low to non-expansive soils, through selective grading and placement of the clayey soils at depths greater than 2 feet below the proposed building pad grades, or a combination of both of these mitigation measures.

Soluble Sulfates

The results of the soluble sulfate testing indicate that the selected samples of the on-site soils contain negligible concentrations of soluble sulfates, in accordance with American Concrete Institute (ACI) guidelines. Therefore, specialized concrete mix designs are not considered to be necessary, with regard to sulfate protection purposes. It is, however, recommended that additional soluble sulfate testing be conducted at the completion of rough grading to verify the soluble sulfate concentrations of the soils which are present at pad grade within the building area.

Shrinkage/Subsidence

Removal and recompaction of the near surface native soils is estimated to result in an average shrinkage of 10 to 15 percent. Minor ground subsidence is expected to occur in the soils below the zone of removal, due to settlement and machinery working. The subsidence is estimated to be 0.1± feet. This estimate may be used for grading in areas that are underlain by native alluvial soils.

These estimates are based on previous experience and the subsurface conditions encountered at the boring locations. The actual amount of subsidence is expected to be variable and will be dependant on the type of machinery used, repetitions of use, and dynamic effects, all of which are difficult to assess precisely.

Grading and Foundation Plan Review

Grading and foundation plans were not available at the time of this report. It is therefore recommended that we be provided with copies of the preliminary grading and foundation plans, when they become available, for review with regard to the conclusions, recommendations, and assumptions contained within this report.

6.3 Site Grading Recommendations

The grading recommendations presented below are based on the subsurface conditions encountered at the boring locations and our understanding of the proposed development. We recommend that all grading activities be completed in accordance with the Grading Guide Specifications included as Appendix D of this report, unless superseded by site-specific recommendations presented below.

As discussed in Section 6.2 of this report it is recommended that a supplemental geotechnical investigation be performed at the subject site, in order to more completely characterize the subsurface conditions in areas of the site which could not be adequately explored during subsurface exploration for this report. Due to the ongoing operations of the Safeway distribution center, it was not feasible to adequately explore proposed Buildings 1, 2, 3, and 8 to thoroughly characterize the subsurface conditions in these proposed building areas. Therefore, it should be understood that the subsurface conditions, in areas lacking borings and or exploratory trenches, may vary significantly than those described in this report.

Site Stripping and Demolition

Demolition of the existing structures and pavements will be necessary in order to facilitate the construction of the proposed development. Demolition should include all foundations, floor slabs, utilities and any other subsurface improvements that will not remain in place with the new development. Concrete and asphalt debris may be crushed to a maximum 2-inch particle size, mixed with the on-site soils, and reused as compacted structural fill. It may also be feasible to crush these materials for use as crushed miscellaneous base (CMB).

Based on visual observations, at least two underground storage tanks (UST) are present in the central portion of the site. Any USTs and any associated product lines should be removed from the proposed building areas in their entirety. Any pea gravel present in these excavations and any soils disturbed during demolition should also be removed and replaced with compacted structural fil materials

Initial site stripping should include removal of any surficial vegetation from the unpaved areas of the site. This should include any weeds, grasses, shrubs, and trees. Root balls associated with the trees should be removed in their entirety, and the resultant excavation should be backfilled with compacted structural fill soils. The actual extent of site stripping should be determined in the field by the geotechnical engineer, based on the organic content and stability of the materials encountered.

Treatment of Existing Soils: Building Pads

Remedial grading should be performed within the proposed building areas in order to remove the artificial fill materials, the upper portion of the alluvial soils, and any soils disturbed during the demolition of the existing site improvements. Based on conditions encountered at the boring locations, the existing soils within the proposed building areas are recommended to be overexcavated to a depth of at least 5 feet below the proposed building pad subgrade elevation and to a depth of at least 5 feet below existing grade, whichever is greater. The depth of the overexcavation should also extend to a depth sufficient to remove all artificial fill soils and any soils disturbed during demolition. Within the influence zones of the new foundations, the overexcavation should extend to a depth of at least 3 feet below proposed foundation bearing grade.

The overexcavation areas should extend at least 5 feet beyond the building perimeters and foundations, and to an extent equal to the depth of fill below the new foundations. If the proposed structures incorporate any exterior columns (such as for a canopy or overhang) the overexcavation should also encompass these areas.

Following completion of the overexcavation, the subgrade soils within the building areas should be evaluated by the geotechnical engineer to verify their suitability to serve as the structural fill subgrade, as well as to support the foundation loads of the new structures. This evaluation should include proofrolling and probing to identify any soft, loose or otherwise unstable soils that must be removed. Some localized areas of deeper excavation may be required if additional fill materials or loose, porous, or low density native soils are encountered at the base of the overexcavation.

A few of the borings encountered very moist silty and clayey soils within the upper 5 to 8± feet at the boring locations. Similar soils may be present at other locations that were not explored by the borings. Due to their relatively high moisture contents, these soils may be unstable when exposed during grading. Air drying of these soils may be feasible, if grading occurs during a period of relatively dry weather. Alternatively, additional overexcavation to remove unstable, very moist, fine-grained soils could also be performed if drying of these soils cannot be achieved within a reasonable timeframe.

After a suitable overexcavation subgrade has been achieved, the exposed soils should be scarified to a depth of at least 12 inches, moisture conditioned to 2 to 4 percent above the optimum moisture content. The subgrade soils should then be recompacted to at least 90 percent of the ASTM D-1557 maximum dry density. The previously excavated soils may then be replaced as compacted structural fill.

As discussed in the previous section, the majority of the near-surface soils are considered to possess very low to non-expansive expansion potentials. However, clayey soils were encountered within the upper 5± feet at a few of the boring locations, possessing medium expansion potentials. Clayey soils should be thoroughly blended with very low to non-expansive soils such that the resultant mixture possesses a very low expansion potential (EI < 20) and/or selectively placed within compacted fills at depths greater than 2 feet below the proposed finished building pad grade.

Treatment of Existing Soils: Retaining Walls and Site Walls

The existing soils within the areas of any proposed retaining and site walls should be overexcavated to a depth of 2 feet below foundation bearing grade and replaced as compacted structural fill as discussed above for the proposed building pad. Any undocumented fill soils within any of these foundation areas should be removed in their entirety. The overexcavation subgrade soils should be evaluated by the geotechnical engineer prior to scarifying, moisture conditioning, and recompacting the upper 12 inches of exposed subgrade soils, as discussed for the building areas. The previously excavated soils may then be replaced as compacted structural fill.

Treatment of Existing Soils: Parking and Drive Areas

Based on economic considerations, overexcavation of the existing soils in the new parking and drive areas is not considered warranted, with the exception of areas where lower strength, or unstable, soils are identified by the geotechnical engineer during grading. Subgrade preparation in the new parking and drive areas should initially consist of removal of all soils disturbed during stripping and demolition operations.

The geotechnical engineer should then evaluate the subgrade to identify any areas of additional unsuitable soils. Any such materials should be removed to a level of firm and unyielding soil. The exposed subgrade soils should then be scarified to a depth of 12± inches, moisture conditioned to at least 2 to 4 percent above the optimum moisture content, and recompacted to at least 90 percent of the ASTM D-1557 maximum dry density. Based on the presence of variable strength surficial soils throughout the site, it is expected that some isolated areas of additional overexcavation may be required to remove zones of lower strength, unsuitable soils.

The grading recommendations presented above for the proposed parking area assume that the owner and/or developer can tolerate minor amounts of settlement within the proposed parking and drive areas. The grading recommendations presented above do not completely mitigate the extent of undocumented fill soils or compressible native alluvium in the parking and drive areas. As such, settlement and associated pavement distress could occur. Typically, repair of such distressed areas involves significantly lower costs than completely mitigating these soils at the time of construction. If the owner cannot tolerate the risk of such settlements, the parking and drive areas should be graded in a manner similar to that described for the building area.

Fill Placement

- Fill soils should be placed in thin (6± inches), near-horizontal lifts, moisture conditioned to 2 to 4 percent above the optimum moisture content, and compacted.
- On-site soils may be used for fill provided they are cleaned of any debris to the satisfaction of the geotechnical engineer.
- All grading and fill placement activities should be completed in accordance with the requirements of the 2013 CBC and the grading code of the City of El Monte.
- All fill soils should be compacted to at least 90 percent of the ASTM D-1557 maximum dry density. Fill soils should be well mixed.
- Compaction tests should be performed periodically by the geotechnical engineer as random verification of compaction and moisture content. These tests are intended to aid the contractor. Since the tests are taken at discrete locations and depths, they may not be indicative of the entire fill and therefore should not relieve the contractor of his responsibility to meet the job specifications.

Imported Structural Fill

All imported structural fill should consist of very low expansive (EI < 20), well graded soils possessing at least 10 percent fines (that portion of the sample passing the No. 200 sieve). Additional specifications for structural fill are presented in the Grading Guide Specifications, included as Appendix D.

Utility Trench Backfill

In general, all utility trench backfill soils should be compacted to at least 90 percent of the ASTM D-1557 maximum dry density. As an alternative, a clean sand (minimum Sand Equivalent of 30) may be placed within trenches and compacted in place (jetting or flooding is not recommended). It is recommended that materials in excess of 3 inches in size not be used for utility trench backfill. Compacted trench backfill should conform to the requirements of the local grading code, and more restrictive requirements may be indicated by City of El Monte. All utility trench backfills should be witnessed by the geotechnical engineer. The trench backfill soils should be compaction tested where possible; probed and visually evaluated elsewhere.

Utility trenches which parallel a footing, and extending below a 1h:1v plane projected from the outside edge of the footing should be backfilled with structural fill soils, compacted to at least 90 percent of the ASTM D-1557 standard. Pea gravel backfill should not be used for these trenches.

6.4 Construction Considerations

Excavation Considerations

The near surface soils generally consist of fine sands and silty sands with some areas of fine sandy silts, fine sandy clays, and clayey fine sands. Most of these materials will likely be subject to caving within shallow excavations. Where caving occurs within shallow excavations, flattened excavation slopes may be sufficient to provide excavation stability. On a preliminary basis, the inclination of temporary slopes should not exceed 2h:1v. Deeper excavations, such as those made for UST removals, may require some form of external stabilization such as shoring or bracing. Maintaining adequate moisture content within the near-surface soils will improve excavation stability. All excavation activities on this site should be conducted in accordance with Cal-OSHA regulations.

Expansive Soils

The majority of the near-surface soils consist of sands and silty sands and are considered to be very low to non-expansive. However, clayey soils with medium expansion potentials were encountered at a few of the boring locations. We recommend that any clayey soils be thoroughly blended with very low to non-expansive soils such that the resultant mixture possesses a very low expansion potential (EI < 20) and/or be placed at depths of greater than 2 feet below the proposed building pad grade or pavement subgrades. Thorough blending and/or selective placement is expected to result in building pads with very low expansion potentials. The project budget should include a contingency for the recommended blending and/or selective handling of expansive clay materials.

Groundwater

The static groundwater table at this site is considered to be present at a depth in excess of 50± feet. Therefore, groundwater is not expected to impact the grading or foundation construction activities.

6.5 Foundation Design and Construction

Based on the preceding grading recommendations, it is assumed that the new building pads will be underlain by structural fill soils used to replace existing fill and near surface alluvial soils. These new structural fill soils are expected to extend to depths of at least 3 feet below proposed foundation bearing grades, underlain by 1± foot of additional soil that has been densified and moisture conditioned in place. Based on this subsurface profile, the proposed structures may be supported on shallow foundations.

Foundation Design Parameters

New square and rectangular footings may be designed as follows:

- Maximum, net allowable soil bearing pressure: 2,500 lbs/ft^2.

- Minimum wall/column footing width: 14 inches/24 inches.

- Minimum longitudinal steel reinforcement within strip footings: Six (6) No. 5 rebars (3 top and 3 bottom) in strip footings, due to the presence of potentially liquefiable soils.

- Minimum foundation embedment: 12 inches into suitable structural fill soils, and at least 18 inches below adjacent exterior grade. Interior column footings may be placed immediately beneath the floor slab.

- It is recommended that the perimeter building foundations be continuous across all exterior doorways. Any flatwork adjacent to the exterior doors should be doweled into the perimeter foundations in a manner determined by the structural engineer.

The allowable bearing pressures presented above may be increased by 1/3 when considering short duration wind or seismic loads. The minimum steel reinforcement recommended above is based on standard geotechnical practice. Additional rigidity may be necessary for structural considerations, or to resist the effects of the liquefaction-induced differential settlements, as discussed in Section 6.1. The actual design of the foundations should be determined by the structural engineer.

Foundation Construction

The foundation subgrade soils should be evaluated at the time of overexcavation, as discussed in Section 6.3 of this report. It is further recommended that the foundation subgrade soils be evaluated by the geotechnical engineer immediately prior to steel or concrete placement. Soils suitable for direct foundation support should consist of newly placed structural fill, compacted to at least 90 percent of the ASTM D-1557 maximum dry density. Any unsuitable materials should be removed to a depth of suitable bearing compacted structural fill, with the resulting excavations backfilled with compacted fill soils. As an alternative, lean concrete slurry (500 to 1,500 psi) may be used to backfill such isolated overexcavations.

The foundation subgrade soils should also be properly moisture conditioned to 2 to 4 percent above the Modified Proctor optimum, to a depth of at least 12 inches below bearing grade. Since it is typically not feasible to increase the moisture content of the floor slab and foundation subgrade soils once rough grading has been completed, care should be taken to maintain the moisture content of the building pad subgrade soils throughout the construction process.

Estimated Foundation Settlements

Post-construction total and differential settlements of shallow foundations designed and constructed in accordance with the previously presented recommendations are estimated to be less than 1.0 and 0.5 inches, respectively. Differential movements are expected to occur over a 30-foot span, thereby resulting in an angular distortion of less than 0.002 inches per inch. These settlements are in addition to the liquefaction-induced settlements previously discussed in Section 6.1 of this report. However, the likelihood of these two settlements combining is considered remote. The static settlements are expected to occur in a relatively short period of time after the building loads being applied to the foundations, during and immediately subsequent to construction. It should be noted that the projected potential dynamic settlement is related to a major seismic event and a conservative historic high groundwater level.

Lateral Load Resistance

Lateral load resistance will be developed by a combination of friction acting at the base of foundations and slabs and the passive earth pressure developed by footings below grade. The following friction and passive pressure may be used to resist lateral forces:

- Passive Earth Pressure: 300 lbs/ft^3
- Friction Coefficient: 0.30

These are allowable values, and include a factor of safety. When combining friction and passive resistance, the passive pressure component should be reduced by one-third. These values assume that footings will be poured directly against compacted structural fill. The maximum allowable passive pressure is 2500 lbs/ft^2.

6.6 Floor Slab Design and Construction

Subgrades which will support new floor slabs should be prepared in accordance with the recommendations contained in the ***Site Grading Recommendations*** section of this report. Based on the anticipated grading which will occur at this site, the floors of the new structures may be constructed as a conventional slabs-on-grade supported on newly placed structural fill, extending to a depth of at least 5 feet below proposed finished pad grade. Based on geotechnical considerations, the floor slabs may be designed as follows:

- Minimum slab thickness: 5 inches.

- Modulus of Subgrade Reaction: k = 100 psi/in.

- Minimum slab reinforcement: Minimum slab reinforcement: No. 3 bars at 18 inches on-center, in both directions, due to the liquefaction potential of the encountered soils. The actual floor slab reinforcement should be determined by the structural engineer, based on the imposed loading.

- Slab underlayment: If moisture sensitive floor coverings will be used, then a moisture vapor barrier should be constructed below the entire slab area where such moisture sensitive floor coverings are anticipated. The moisture vapor barrier should meet or exceed the Class A rating as defined by ASTM E 1745-97 and have a permeance rating less than 0.01 perms as described in ASTM E 96-95 and ASTM E 154-88. The moisture vapor barrier should be properly constructed in accordance with all applicable manufacturer specifications. Given that a rock free subgrade is anticipated and that a capillary break is not required, sand below the barrier is not required. The need for sand and/or the amount of sand above the moisture vapor barrier should be specified by the structural engineer or concrete contractor. The selection of sand above the barrier is not a geotechnical engineering issue and hence outside our purview.

- Moisture condition the floor slab subgrade soils to 2 to 4 percent above the Modified Proctor optimum moisture content, to a depth of 12 inches. The moisture content of the floor slab subgrade soils should be verified by the geotechnical engineer within 24 hours prior to concrete placement.

- Proper concrete curing techniques should be utilized to reduce the potential for slab curling or the formation of excessive shrinkage cracks.

The actual design of the floor slab should be completed by the structural engineer to verify adequate thickness and reinforcement. The steel reinforcement recommendations presented above are based on standard geotechnical practice, given the magnitude of predicted liquefaction-induced settlements, and the structure type proposed for the site. Additional rigidity may be necessary for structural considerations, or to resist the effects of the liquefaction-induced differential settlements discussed in Section 6.1.

6.7 Retaining Wall Design and Construction

New retaining walls are expected to be necessary in the truck court areas. Additionally, although not indicated on the site plan, the proposed development may require some small retaining walls (less than 5± feet in height) to facilitate the new site grades and in loading dock areas.

Retaining Wall Design Parameters

Based on the soil conditions encountered at the boring locations, the following parameters may be used in the design of new retaining walls for this site. We have provided parameters assuming the use of on-site soils for retaining wall backfill. The near surface soils generally consist of fine sands and silty fine sands with some areas containing fine sandy silty, fine sandy clays and clayey fine sands. Expansive clayey soils should not be used as retaining wall backfill. Based on their classifications, the sand and silty sand materials are expected to possess a friction angle of at least 30 degrees when compacted to 90 percent of the ASTM-1557 maximum dry density.

If desired, SCG could provide design parameters for an alternative select backfill material behind the retaining walls. The use of select backfill material could result in lower lateral earth pressures. In order to use the design parameters for the imported select fill, this material must be placed within the entire active failure wedge. This wedge is defined as extending from the heel of the retaining wall upwards at an angle of approximately 60° from horizontal. If select backfill material behind the retaining wall is desired, SCG should be contacted for supplementary recommendations.

RETAINING WALL DESIGN PARAMETERS

Design Parameter		Soil Type: On-Site Sands and Silty Sands
Internal Friction Angle (ϕ)		30°
Unit Weight		120 lbs/ft^3
Equivalent Fluid Pressure:	Active Condition (level backfill)	40 lbs/ft^3
	Active Condition (2h:1v backfill)	65 lbs/ft^3
	At-Rest Condition (level backfill)	60 lbs/ft^3

Regardless of the backfill type, the walls should be designed using a soil-footing coefficient of friction of 0.30 and an equivalent passive pressure of 300 lbs/ft^3. The structural engineer should incorporate appropriate factors of safety in the design of the retaining walls.

The active earth pressure may be used for the design of retaining walls that do not directly support structures or support soils that in turn support structures and which will be allowed to deflect. The at-rest earth pressure should be used for walls that will not be allowed to deflect such as those which will support foundation bearing soils, or which will support foundation loads directly.

Where the soils on the toe side of the retaining wall are not covered by a "hard" surface such as a structure or pavement, the upper 1 foot of soil should be neglected when calculating passive resistance due to the potential for the material to become disturbed or degraded during the life of the structure.

Seismic Lateral Earth Pressures

In accordance with the 2013 CBC, any retaining walls more than 6 feet in height must be designed for seismic lateral earth pressures. If walls 6 feet or more are required for this site, the geotechnical engineer should be contacted for supplementary seismic lateral earth pressure recommendations.

Retaining Wall Foundation Design

The retaining wall foundations should be supported within newly placed compacted structural fill, extending to a depth of at least 2 feet below the proposed bearing grade. Foundations to support new retaining walls should be designed in accordance with the general Foundation Design Parameters presented in a previous section of this report.

Backfill Material

On-site soils may be used to backfill the retaining walls. However, all backfill material placed within 3 feet of the back wall face should have a particle size no greater than 3 inches. The retaining wall backfill materials should be well graded.

It is recommended that a properly installed prefabricated drainage composite such as the MiraDRAIN 6000XL (or approved equivalent), which is specifically designed for use behind retaining walls be used. If the drainage composite material is not covered by an impermeable surface, such as a structure or pavement, a 12-inch thick layer of a low permeability soil should be placed over the backfill to reduce surface water migration to the underlying soils. The drainage composite should be separated from the backfill soils by a suitable geotextile, approved by the geotechnical engineer.

All retaining wall backfill should be placed and compacted under engineering controlled conditions in the necessary layer thicknesses to ensure an in-place density between 90 and 93 percent of the maximum dry density as determined by the Modified Proctor test (ASTM D1557). Care should be taken to avoid over-compaction of the soils behind the retaining walls, and the use of heavy compaction equipment should be avoided.

Subsurface Drainage

As previously indicated, the retaining wall design parameters are based upon drained backfill conditions. Consequently, some form of permanent drainage system will be necessary in conjunction with the appropriate backfill material. Subsurface drainage may consist of either:

- A weep hole drainage system typically consisting of a series of 4-inch diameter holes in the wall situated slightly above the ground surface elevation on the exposed side of the wall and at an approximate 8-foot on-center spacing. The weep holes should include a one cubic foot gravel pocket surrounded by a suitable geotextile at each weep hole location.

- A 4-inch diameter perforated pipe surrounded by 2 cubic feet of gravel per linear foot of drain placed behind the wall, above the retaining wall footing. The gravel layer should be wrapped in a suitable geotextile fabric to reduce the potential for migration of fines. The footing drain should be extended to daylight or tied into a storm drainage system.

6.8 Pavement Design Parameters

Site preparation in the pavement area should be completed as previously recommended in the ***Site Grading Recommendations*** section of this report. The subsequent pavement recommendations assume proper drainage and construction monitoring, and are based on either PCA or CALTRANS design parameters for a twenty (20) year design period. However, these designs also assume a routine pavement maintenance program to obtain the anticipated 20-year pavement service life.

Pavement Subgrades

It is anticipated that the new pavements will be primarily supported on a layer of compacted structural fill, consisting of scarified, thoroughly moisture conditioned and recompacted on-site soils. The near-surface soils generally consist of silty sands and sands with some areas containing fine sandy silty, fine sandy clays and clayey fine sands. The on-site soils are considered to possess good pavement support characteristics with estimated R-values of 40 to 50. Since R-value testing was not included in the scope of services for this project, the subsequent pavement design is based upon an assumed R-value of 40. Any fill material imported to the site should have support characteristics equal to or greater than that of the on-site soils and be placed and compacted under engineering controlled conditions. It is recommended that R-value testing be performed after completion of rough grading.

Asphaltic Concrete

Presented below are the recommended thicknesses for new flexible pavement structures consisting of asphaltic concrete over a granular base. The pavement designs are based on the traffic indices (TI's) indicated. The client and/or civil engineer should verify that these TI's are representative of the anticipated traffic volumes. If the client and/or civil engineer determine that the expected traffic volume will exceed the applicable traffic index, we should be contacted for supplementary recommendations. The design traffic indices equate to the following approximate daily traffic volumes over a 20 year design life, assuming six operational traffic days per week.

Traffic Index	**No. of Heavy Trucks per Day**
4.0	0
5.0	1
6.0	3
7.0	11
8.0	35

For the purpose of the traffic volumes indicated above, a truck is defined as a 5-axle tractor trailer unit with one 8-kip axle and two 32-kip tandem axles. All of the traffic indices allow for 1,000 automobiles per day.

ASPHALT PAVEMENTS (R = 40)					
Materials	**Thickness (inches)**				
	Auto Parking (TI = 4.0)	Auto Drive Lanes (TI = 5.0)	Truck Traffic		
			(TI = 6.0)	(TI = 7.0)	(TI = 8.0)
Asphalt Concrete	3	3	3½	4	5
Aggregate Base	3	4	6	7	8
Compacted Subgrade	12	12	12	12	12

The aggregate base course should be compacted to at least 95 percent of the ASTM D-1557 maximum dry density. The asphaltic concrete should be compacted to at least 95 percent of the Marshall maximum density, as determined by ASTM D-2726. The aggregate base course may consist of crushed aggregate base (CAB) or crushed miscellaneous base (CMB), which is a recycled gravel, asphalt and concrete material. The gradation, R-Value, Sand Equivalent, and Percentage Wear of the CAB or CMB should comply with appropriate specifications contained in the current edition of the "Greenbook" Standard Specifications for Public Works Construction.

Portland Cement Concrete

The preparation of the subgrade soils within concrete pavement areas should be performed as previously described for proposed asphalt pavement areas. The minimum recommended thicknesses for the Portland Cement Concrete pavement sections are as follows:

PORTLAND CEMENT CONCRETE PAVEMENTS				
Materials	**Thickness (inches)**			
	Auto Parking & Drives (TI = 5.0)	Truck Traffic		
		(TI = 6.0)	(TI = 7.0)	(TI = 8.0)
PCC	5	5½	6½	8
Compacted Subgrade (95% Relative Compaction)	12	12	12	12

The concrete should have a 28-day compressive strength of at least 3,000 psi. The maximum joint spacing within all of the PCC pavements is recommended to be equal to or less than 30 times the pavement thickness.

7.0 GENERAL COMMENTS

This report has been prepared as an instrument of service for use by the client, in order to aid in the evaluation of this property and to assist the architects and engineers in the design and preparation of the project plans and specifications. This report may be provided to the contractor(s) and other design consultants to disclose information relative to the project. However, this report is not intended to be utilized as a specification in and of itself, without appropriate interpretation by the project architect, civil engineer, and/or structural engineer. The reproduction and distribution of this report must be authorized by the client and Southern California Geotechnical, Inc. Furthermore, any reliance on this report by an unauthorized third party is at such party's sole risk, and we accept no responsibility for damage or loss which may occur. The client(s)' reliance upon this report is subject to the Engineering Services Agreement, incorporated into our proposal for this project.

The analysis of this site was based on a subsurface profile interpolated from limited discrete soil samples. While the materials encountered in the project area are considered to be representative of the total area, some variations should be expected between boring locations and sample depths. If the conditions encountered during construction vary significantly from those detailed herein, we should be contacted immediately to determine if the conditions alter the recommendations contained herein.

This report has been based on assumed or provided characteristics of the proposed development. It is recommended that the owner, client, architect, structural engineer, and civil engineer carefully review these assumptions to ensure that they are consistent with the characteristics of the proposed development. If discrepancies exist, they should be brought to our attention to verify that they do not affect the conclusions and recommendations contained herein. We also recommend that the project plans and specifications be submitted to our office for review to verify that our recommendations have been correctly interpreted.

The analysis, conclusions, and recommendations contained within this report have been promulgated in accordance with generally accepted professional geotechnical engineering practice. No other warranty is implied or expressed.

8.0 REFERENCES

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Southern California Earthquake Center (SCEC), University of Southern California, "Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Liquefaction in California," Committee formed 1997.

Tokimatsu K., and Seed, H. B., "Evaluation of Settlements in Sands Due to Earthquake Shaking," Journal of the Geotechnical Engineering Division, American society of Civil Engineers, Volume 113, No. 8, August 1987, pp. 861-878.

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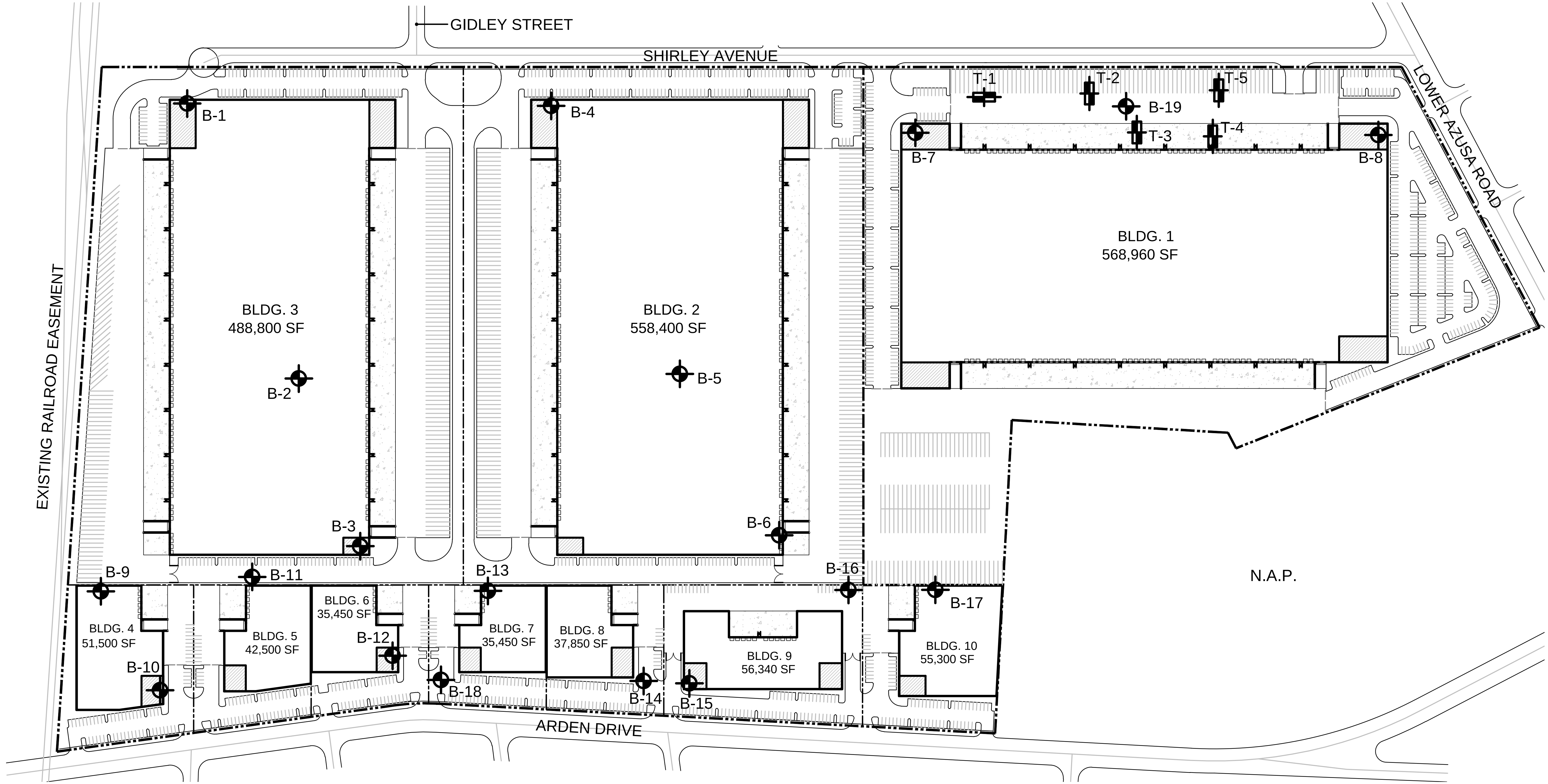
Youd, T. L. and Idriss, I. M. (Editors), "Proceedings of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils," Salt Lake City, UT, January 5-6 1996, NCEER Technical Report NCEER-97-0022, Buffalo, NY.

APPENDIX A

SOURCE: LOS ANGELES COUNTY
THOMAS GUIDE, 2013

SITE LOCATION MAP	
PROPOSED COMMERCIAL INDUSTRIAL DEVELOPMENT	
LOS ANGELES, CALIFORNIA	
SCALE: 1" = 2400'	SOUTHERN CALIFORNIA GEOTECHNICAL
DRAWN: JL CHKD: DWN	
SCG PROJECT 15G227-1	
PLATE 1	



GEOTECHNICAL LEGEND

APPROXIMATE BORING LOCATION

APPROXIMATE TRENCH LOCATION

NOTE: BASE SITE MAP PREPARED BY HPA ARCHITECTS.

BORING AND TRENCH LOCATION PLAN

PROPOSED COMMERCIAL/INDUSTRIAL DEVELOPMENT

EL MONTE, CALIFORNIA

SCALE: 1" = 200'

DRAWN: MRM
CHKD: JAS

SCG PROJECT
15G227-1

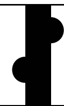
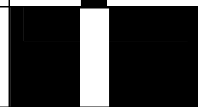
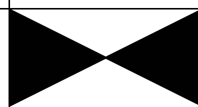
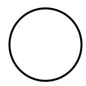
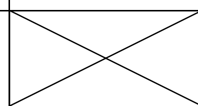
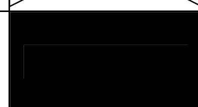
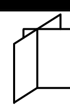
PLATE 2

SoCalGeo

SOUTHERN
CALIFORNIA
GEOTECHNICAL

APPENDIX B

BORING LOG LEGEND

SAMPLE TYPE	GRAPHICAL SYMBOL	SAMPLE DESCRIPTION
AUGER		SAMPLE COLLECTED FROM AUGER CUTTINGS, NO FIELD MEASUREMENT OF SOIL STRENGTH. (DISTURBED)
CORE		ROCK CORE SAMPLE: TYPICALLY TAKEN WITH A DIAMOND-TIPPED CORE BARREL. TYPICALLY USED ONLY IN HIGHLY CONSOLIDATED BEDROCK.
GRAB		SOIL SAMPLE TAKEN WITH NO SPECIALIZED EQUIPMENT, SUCH AS FROM A STOCKPILE OR THE GROUND SURFACE. (DISTURBED)
CS		CALIFORNIA SAMPLER: 2-1/2 INCH I.D. SPLIT BARREL SAMPLER, LINED WITH 1-INCH HIGH BRASS RINGS. DRIVEN WITH SPT HAMMER. (RELATIVELY UNDISTURBED)
NSR		NO RECOVERY: THE SAMPLING ATTEMPT DID NOT RESULT IN RECOVERY OF ANY SIGNIFICANT SOIL OR ROCK MATERIAL.
SPT		STANDARD PENETRATION TEST: SAMPLER IS A 1.4 INCH INSIDE DIAMETER SPLIT BARREL, DRIVEN 18 INCHES WITH THE SPT HAMMER. (DISTURBED)
SH		SHELBY TUBE: TAKEN WITH A THIN WALL SAMPLE TUBE, PUSHED INTO THE SOIL AND THEN EXTRACTED. (UNDISTURBED)
VANE		VANE SHEAR TEST: SOIL STRENGTH OBTAINED USING A 4 BLADED SHEAR DEVICE. TYPICALLY USED IN SOFT CLAYS-NO SAMPLE RECOVERED.

COLUMN DESCRIPTIONS

DEPTH:	Distance in feet below the ground surface.
SAMPLE:	Sample Type as depicted above.
BLOW COUNT:	Number of blows required to advance the sampler 12 inches using a 140 lb hammer with a 30-inch drop. 50/3” indicates penetration refusal (>50 blows) at 3 inches. WH indicates that the weight of the hammer was sufficient to push the sampler 6 inches or more.
POCKET PEN.:	Approximate shear strength of a cohesive soil sample as measured by pocket penetrometer.
GRAPHIC LOG:	Graphic Soil Symbol as depicted on the following page.
DRY DENSITY:	Dry density of an undisturbed or relatively undisturbed sample in lbs/ft^3.
MOISTURE CONTENT:	Moisture content of a soil sample, expressed as a percentage of the dry weight.
LIQUID LIMIT:	The moisture content above which a soil behaves as a liquid.
PLASTIC LIMIT:	The moisture content above which a soil behaves as a plastic.
PASSING #200 SIEVE:	The percentage of the sample finer than the #200 standard sieve.
UNCONFINED SHEAR:	The shear strength of a cohesive soil sample, as measured in the unconfined state.

SOIL CLASSIFICATION CHART

MAJOR DIVISIONS			SYMBOLS		TYPICAL DESCRIPTIONS
			GRAPH	LETTER	
COARSE GRAINED SOILS	GRAVEL AND GRAVELLY SOILS	CLEAN GRAVELS		GW	WELL-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
		(LITTLE OR NO FINES)		GP	POORLY-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
	MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	GRAVELS WITH FINES		GM	SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES
		(APPRECIABLE AMOUNT OF FINES)		GC	CLAYEY GRAVELS, GRAVEL - SAND - CLAY MIXTURES
MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	SAND AND SANDY SOILS	CLEAN SANDS		SW	WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
		(LITTLE OR NO FINES)		SP	POORLY-GRADED SANDS, GRAVELLY SAND, LITTLE OR NO FINES
	MORE THAN 50% OF COARSE FRACTION PASSING ON NO. 4 SIEVE	SANDS WITH FINES		SM	SILTY SANDS, SAND - SILT MIXTURES
		(APPRECIABLE AMOUNT OF FINES)		SC	CLAYEY SANDS, SAND - CLAY MIXTURES
FINE GRAINED SOILS	SILTS AND CLAYS	LIQUID LIMIT LESS THAN 50		ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS OR CLAYEY SILTS WITH SLIGHT PLASTICITY
				CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
				OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
MORE THAN 50% OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	SILTS AND CLAYS	LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILTY SOILS
				CH	INORGANIC CLAYS OF HIGH PLASTICITY
				OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC SOILS				PT	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-1

JOB NO.: 15G227
PROJECT: Proposed C/I Development
LOCATION: El Monte, California

DRILLING DATE: 12/18/15
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Matt Manni

WATER DEPTH: Dry
CAVE DEPTH: 15.5 feet
READING TAKEN: At Completion

SURFACE ELEVATION: --- MSL

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
0					5± inches Asphaltic concrete, 1± inch Aggregate base							
					<u>FILL:</u> Brown to Dark Gray Brown Silty fine to medium Sand, trace coarse Sand, little Silt, mottled, very loose to loose-moist							
	X	5					8					
	X	2					8					
5												
					<u>POSSIBLE FILL:</u> Brown Silty fine to medium Sand, trace coarse Sand, loose-moist							
	X	6					9					
					<u>ALLUVIUM:</u> Gray Brown fine to coarse Sand, medium dense-dry to damp							
	X	11					2			3		
10												
	X	18			@ 13½ to 15 feet, little coarse Sand, trace fine Gravel		2			3		
15												
					Brown Silty fine Sand with thinly interbedded fine Sandy Silt lenses, trace Iron oxide staining, loose to medium dense-moist							
	X	10					13			36		
20												
					Light Brown to Gray Brown Silty fine Sand, trace Iron oxide staining, medium dense-damp							
	X	15					6			14		
25												
					Gray Brown fine to coarse Sand, little fine to coarse Gravel, occasional Cobbles, dense-dry to damp							
	X	35					2					
30												
	X	46					3					

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-1

JOB NO.: 15G227
PROJECT: Proposed C/I Development
LOCATION: El Monte, California

DRILLING DATE: 12/18/15
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Matt Manni

WATER DEPTH: Dry
CAVE DEPTH: 15.5 feet
READING TAKEN: At Completion

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION (Continued)	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
					Gray Brown fine to coarse Sand, little fine to coarse Gravel, occasional Cobbles, dense-dry to damp							
40	X	48					3					
45	X	46					3					
					Gray Brown Silty fine Sand interbedded with fine Sandy Silt, trace medium Sand, trace Iron oxide staining, trace Clay clasts, medium dense to dense-moist							
50	X	29					9			26		
					Boring Terminated at 50'							

TBL 15G227.GPJ SOCALGEO.GDT 2/3/16

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-2

JOB NO.: 15G227
PROJECT: Proposed C/I Development
LOCATION: El Monte, California

DRILLING DATE: 1/7/16
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Matt Manni

WATER DEPTH: Dry
CAVE DEPTH: 8 feet
READING TAKEN: At Completion

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: --- MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
					5± inches Asphaltic concrete, 4± inches Aggregate base							
		16			FILL: Brown Silty fine to medium Sand, trace coarse Sand, trace fine to coarse Gravel, mottled, loose to medium dense-moist to very moist	115	9					
		10				113	17					
5		19				118	11					
					FILL: Dark Gray Silty fine to medium Sand, trace coarse Sand, trace fine to coarse Gravel, mottled, medium dense-moist							
		18				116	12					
		12			ALLUVIUM: Gray Brown Silty fine Sand, trace medium Sand, trace Iron oxide staining, loose-very moist	104	21					
10												
					Light Gray Brown fine Sandy Silt, trace Iron oxide staining, medium dense-very moist							
		13					35					
15					Light Gray Brown fine Sand, trace Silt, medium dense-damp		6					

Boring Terminated at 15'

TBL 15G227.GPJ SOCALGEO.GDT 2/3/16

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-3

JOB NO.: 15G227	DRILLING DATE: 1/6/16	WATER DEPTH: Dry
PROJECT: Proposed C/I Development	DRILLING METHOD: Hollow Stem Auger	CAVE DEPTH: 12 feet
LOCATION: El Monte, California	LOGGED BY: Matt Manni	READING TAKEN: At Completion

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: --- MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
					3± inches Asphaltic concrete, 2± inches Aggregate base							
		7			FILL: Light Gray Brown fine Sand, little Silt, trace Iron oxide staining, loose-very moist	84	15					EI = 58 @ 0 to 5'
		8			FILL: Dark Brown fine Sandy Clay, trace Iron oxide staining, slightly porous, loose-very moist	71	39					
5		11			@ 5 to 5½ feet, trace calcareous veining	91	31					
		12			ALLUVIUM: Light Brown Silty fine Sand to fine Sandy Silt, trace Iron oxide staining, loose-very moist	96	18					
10		19			Light Gray Brown fine Sand, trace Silt, trace Iron oxide staining, medium dense-damp to moist	95	8					
15		26			Light Gray Brown fine to coarse Sand, trace Silt, trace fine Gravel, medium dense-damp		4					
20		14			Brown Silty fine Sand, trace medium Sand, trace Iron oxide staining, medium dense-moist		12					
					Boring Terminated at 20'							

TBL 15G227.GPJ SOCALGEO.GDT 2/3/16

BORING NO. B-4

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

JOB NO.: 15G227 | DRILLING DATE: 12/18/15 | WATER DEPTH: Dry
PROJECT: Proposed C/I Development | DRILLING METHOD: Hollow Stem Auger | CAVE DEPTH: 8.5 feet
LOCATION: El Monte, California | LOGGED BY: Matt Manni | READING TAKEN: At Completion

DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: --- MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
					6± inches Asphaltic concrete, 3± inches Aggregate base							
	X	30			FILL: Brown fine to coarse Sand, trace Silt, trace fine Gravel, trace Asphaltic concrete fragments, medium dense-damp to moist	111	5					
	X	23			ALLUVIUM: Light Gray Brown fine to medium Sand, trace coarse Sand, trace Silt, medium dense-moist	101	8					
5	X	26			Light Gray Brown fine to coarse Sand, little fine to coarse Gravel, occasional Cobbles, medium dense-dry to damp	117	2					
	X	30					2					Disturbed Sample
10	X	16			Red Brown fine to medium Sand, little Silt, some Iron oxide staining, loose-damp	99	6					
15	X	38			Light Gray fine to coarse Sand, trace fine to coarse Gravel, medium dense-dry to damp	112	2					
20	X	51				104	2					

Boring Terminated at 20'

TBL 15G227.GPJ SOCALGEO.GDT 2/3/16

TEST BORING LOG **PLATE B-4**

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-5

JOB NO.: 15G227
PROJECT: Proposed C/I Development
LOCATION: El Monte, California

DRILLING DATE: 1/7/16
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Matt Manni

WATER DEPTH: Dry
CAVE DEPTH: 9 feet
READING TAKEN: At Completion

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: --- MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
					5± inches Asphaltic concrete, 24± inches Aggregate base							
	X											
					FILL: Gray Brown Silty fine to medium Sand, trace coarse Sand, trace fine to coarse Gravel, mottled, medium dense-damp to moist							
	X	12					8					
5					FILL: Dark Gray fine Sandy Silt, trace Clay, trace medium to coarse Sand, trace fine Gravel, mottled, medium dense-very moist		19					
	X	12			ALLUVIUM: Brown Silty fine to medium Sand, trace coarse Sand, trace Iron oxide staining, loose to medium dense-very moist		12					
	X	8					15					
10												
	X	15					17					
15												

Boring Terminated at 15'

TBL 15G227.GPJ SOCALGEO.GDT 2/3/16

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-6

JOB NO.: 15G227
PROJECT: Proposed C/I Development
LOCATION: El Monte, California

DRILLING DATE: 12/21/15
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Matt Manni

WATER DEPTH: Dry
CAVE DEPTH: 13 feet
READING TAKEN: At Completion

SURFACE ELEVATION: --- MSL

DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	DESCRIPTION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
				4± inches Asphaltic concrete, no discernible Aggregate base							
	X	11		FILL: Dark Gray Brown Silty fine to medium Sand, trace coarse Sand, trace fine Gravel, mottled, loose-very moist	103	14					
	X	16		ALLUVIUM: Light Brown Silty fine Sand, trace medium to coarse Sand, trace Iron oxide staining, medium dense-damp to moist	111	8					
5	X	19			115	8					
	X	21		Light Brown fine Sand, trace to little Silt, trace medium to coarse Sand, little Iron oxide staining, medium dense-damp to moist	107	6					
10	X	17			113	11					
15	X	11				12					
20	X	14				10					
				Boring Terminated at 20'							

TBL 15G227.GPJ SOCALGEO.GDT 2/3/16

BORING NO. B-7

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

JOB NO.: 15G227
PROJECT: Proposed C/I Development
LOCATION: El Monte, California

DRILLING DATE: 1/7/16
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Matt Manni

WATER DEPTH: Dry
CAVE DEPTH: 10 feet
READING TAKEN: At Completion

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: --- MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
					4± inches Asphaltic concrete, 4± inches Aggregate base							
		16			FILL: Brown Silty fine to medium Sand, trace coarse Sand, mottled, medium dense-moist to very moist	115	9					
		15			@ 3 to 4 feet, Red Brown	111	14					
5		10			ALLUVIUM: Brown Silty fine Sand, trace medium Sand, loose-damp	107	4					
		10			Light Gray Brown fine Sand, trace Silt, trace medium to coarse Sand, loose-damp	95	3					
					Brown fine Sandy Silt, loose-very moist							
		12				89	29					
10					Light Gray Brown fine Sand, trace medium Sand, trace Silt, loose-very moist							
					Gray Brown fine Sandy Silt, trace Clay, trace Iron oxide staining, medium dense-very moist							
		11					26					
15												
					Gray Brown fine to coarse Sand, trace fine to coarse Gravel, trace Silt, medium dense-damp							
		24					3					
					Gray Brown Silty fine Sand, trace Iron oxide staining, medium dense-moist to very moist		16					
20												
					Boring Terminated at 20'							

TBL 15G227.GPJ SOCALGEO.GDT 2/3/16

BORING NO. B-8

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

JOB NO.: 15G227
PROJECT: Proposed C/I Development
LOCATION: El Monte, California

DRILLING DATE: 1/6/16
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Matt Manni

WATER DEPTH: Dry
CAVE DEPTH: 29 feet
READING TAKEN: At Completion

SURFACE ELEVATION: --- MSL

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
					5± inches Asphaltic concrete, 3± inches Aggregate base							
		6			FILL: Dark Brown fine to medium Sand, trace coarse Sand, little Silt, little fine Gravel, mottled, very loose to loose-moist		8					
		3					8					
5												
		9			ALLUVIUM: Light Brown fine to medium Sand, little Silt, trace coarse Sand, loose-damp to moist		5					
		10					11			50		
10												
					Light Gray Brown fine to medium Sand, trace Silt, little coarse Sand, medium dense-damp							
		12					4			4		
15												
					Gray Brown fine Sandy Silt, trace medium Sand, trace Iron oxide staining, medium dense-moist to very moist							
		18					15			54		
					Light Gray Brown fine Sand, trace medium Sand, trace Silt, medium dense-damp		3			6		
20												
					Light Gray fine to coarse Sand, trace fine Gravel, medium dense-damp							
		12					3			6		
25												
					Gray Brown Silty fine Sand, trace medium Sand, trace Iron oxide staining, medium dense-very moist							
		14					17			36		
30												
					Gray Brown fine to coarse Sand, trace fine to coarse Gravel, trace Silt, trace Iron oxide staining, medium dense-dry to damp							
		27					2					
					Gray Brown Silty fine Sand, trace medium Sand, trace Iron		10			19		

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-8

JOB NO.: 15G227
PROJECT: Proposed C/I Development
LOCATION: El Monte, California

DRILLING DATE: 1/6/16
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Matt Manni

WATER DEPTH: Dry
CAVE DEPTH: 29 feet
READING TAKEN: At Completion

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION (Continued)	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
					oxide staining, medium dense-very moist							
					Gray Brown Silty fine Sand, trace medium Sand, trace Iron oxide staining, medium dense-very moist							
	X	27					13			23		
40												
					Light Gray Brown fine to coarse Sand, trace fine to coarse Gravel, trace Silt, dense-damp							
	X	42					3					
45												
					Light Gray Brown fine Sand, trace medium Sand, trace Silt, trace Iron oxide staining, dense-damp to moist							
	X	39					6					
50												
					Boring Terminated at 50'							

TBL 15G227.GPJ SOCALGEO.GDT 2/3/16

TEST BORING LOG

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-9

JOB NO.: 15G227
PROJECT: Proposed C/I Development
LOCATION: El Monte, California

DRILLING DATE: 1/5/16
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Matt Manni

WATER DEPTH: Dry
CAVE DEPTH:
READING TAKEN: At Completion

SURFACE ELEVATION: --- MSL

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
					<u>FILL:</u> Dark Gray Brown Clayey fine Sand to fine Sandy Clay, trace medium to coarse Sand, mottled, loose-very moist							
	X	8					18					
	X	7					23					
5												
					<u>POSSIBLE FILL:</u> Brown Silty fine Sand, trace Clay, trace Iron oxide staining, medium dense-moist							
	X	16					12					
					<u>ALLUVIUM:</u> Light Gray Brown Silty fine Sand, trace Iron oxide staining, medium dense-damp							
	X	15					6			18		
10												
					Light Gray fine to coarse Sand, trace Silt, trace fine Gravel, medium dense to dense-dry to damp							
	X	30					2					
15												
	X	15			@ 18½ to 20 feet, trace to little Silt		5			6		
20												
					Brown to Orange Brown fine Sandy Silt with interbedded Silty fine Sand lenses, some Iron oxide staining, medium dense-very moist							
	X	12					27			71		
25												
					Light Gray Brown fine to coarse Sand, trace Silt, dense-damp							
	X	33					3					
30												
					Brown fine to medium Sand, trace coarse Sand, little Silt, trace fine Gravel, trace Iron oxide staining, very dense-very moist							
	X	56					12					

TBL 15G227.GPJ SOCALGEO.GDT 2/3/16

JOB NO.: 15G227
PROJECT: Proposed C/I Development
LOCATION: El Monte, California

DRILLING DATE: 1/5/16
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Matt Manni

WATER DEPTH: Dry
CAVE DEPTH:
READING TAKEN: At Completion

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION (Continued)	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
					Brown fine to medium Sand, trace coarse Sand, little Silt, trace fine Gravel, trace Iron oxide staining, very dense-very moist							
					Light Gray Brown fine to coarse Sand, trace fine to coarse Gravel, trace Silt, dense to very dense-damp to moist							
	X	45					4					
40												
	X	70					7					
45												
					Light Brown to Brown fine Sand, very dense-moist							
	X	61					7					
50												
					Boring Terminated at 50'							

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-10

JOB NO.: 15G227
PROJECT: Proposed C/I Development
LOCATION: El Monte, California
DRILLING DATE: 1/6/16
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Matt Manni
WATER DEPTH: Dry
CAVE DEPTH: 8 feet
READING TAKEN: At Completion

SURFACE ELEVATION: --- MSL

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
					3± inches Asphaltic concrete, 3± inches Aggregate base							
	X	19			FILL: Gray Brown Silty fine Sand, medium dense-moist	89	8					
					ALLUVIUM: Light Gray Brown fine Sand, trace to little Silt, trace Iron oxide staining, medium dense-moist							
	X	10			Light Gray Brown Silty fine Sand to fine Sandy Silt, trace medium Sand, trace Iron oxide staining, slightly porous, trace fine root fibers, loose to medium dense-moist to very moist	73	12					
5	X	16				95	13					
	X	21				97	10					
	X	16			Light Gray Brown fine Sand, trace Silt, trace Iron oxide staining, trace fine root fibers, medium dense-dry to damp	92	4					
10												
	X	24					2					
15					Boring Terminated at 15'							

TBL 15G227.GPJ SOCALGEO.GDT 2/3/16

BORING NO. B-11

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

JOB NO.: 15G227
PROJECT: Proposed C/I Development
LOCATION: El Monte, California

DRILLING DATE: 1/6/16
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Matt Manni

WATER DEPTH: Dry
CAVE DEPTH:
READING TAKEN: At Completion

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: --- MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
					4± inches Asphaltic concrete, 3± inches Aggregate base							
	X	6			POSSIBLE FILL: Dark Gray Brown fine Sandy Silt with thin Silty fine Sand layers, little Clay, trace Iron oxide staining, trace calcareous nodules, loose-very moist		32					
	X	5					26					
5												
	X	8			ALLUVIUM: Dark Brown Silty fine Sand, trace Iron oxide staining, slightly porous, loose-very moist		19					
	X	11			Light Gray Brown fine Sand, little Silt, trace Iron oxide staining, medium dense-damp to moist		7					
10												
					Light Gray fine to coarse Sand, trace Silt, trace Iron oxide staining, medium dense-damp							
	X	28					3					
15												
					Boring Terminated at 15'							

TBL 15G227.GPJ SOCALGEO.GDT 2/3/16

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-12

JOB NO.: 15G227
PROJECT: Proposed C/I Development
LOCATION: El Monte, California

DRILLING DATE: 1/6/16
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Matt Manni

WATER DEPTH: Dry
CAVE DEPTH:
READING TAKEN: At Completion

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: --- MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
					3± inches Asphaltic concrete, 2± inches Aggregate base							
	X	4			POSSIBLE FILL: Dark Brown fine Sandy Clay, little Silt, trace Iron oxide staining, trace calcareous veining, slightly porous, loose-very moist		37					
5	X	7					25					
	X	10			ALLUVIUM: Brown Silty fine Sand to fine Sandy Silt, trace Iron oxide staining, loose-very moist		21					
10	X	10			@ 8½ to 10 feet, trace medium Sand, trace Clay		16					
					Brown Silty fine Sand, little medium Sand, trace coarse Sand, trace Iron oxide staining, medium dense-very moist							
15	X	17					15					
					Light Brown fine to medium Sand, trace coarse Sand, trace Iron oxide staining, medium dense-damp							
20	X	28					4					
					Boring Terminated at 20'							

BORING NO. B-13

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

JOB NO.: 15G227
PROJECT: Proposed C/I Development
LOCATION: El Monte, California

DRILLING DATE: 12/21/15
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Matt Manni

WATER DEPTH: Dry
CAVE DEPTH: 12 feet
READING TAKEN: At Completion

SURFACE ELEVATION: --- MSL

DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
					2± inches Asphaltic concrete, 3± inches Aggregate base							
	X	12			ALLUVIUM: Gray Brown Silty fine Sand, trace medium Sand, trace Clay, slightly porous, loose-very moist	96	20					
	X	5			Dark Gray Brown Silty fine Sand to fine Sandy Silt, trace medium Sand, some Clay, trace calcareous nodules, trace Iron oxide staining, loose-very moist	60	29					
5	X	5				61	30					
	X	9			Light Gray Brown Silty fine Sand, trace Iron oxide staining, trace calcareous veining, slightly porous, loose-moist to very moist	91	17					
	X	11				101	12					
10												
					Light Gray fine to medium Sand, trace to little Silt, medium dense-damp to moist							
	X	13					8					
15												
					Gray Brown fine to coarse Sand, trace fine to coarse Gravel, trace Silt, medium dense-damp							
	X	26					3					
20												

Boring Terminated at 20'

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-14

JOB NO.: 15G227
PROJECT: Proposed C/I Development
LOCATION: El Monte, California

DRILLING DATE: 12/21/15
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Matt Manni

WATER DEPTH: Dry
CAVE DEPTH: 7 feet
READING TAKEN: At Completion

SURFACE ELEVATION: --- MSL

DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
					3± inches Asphaltic concrete, 3± inches Aggregate base							
	X	6			FILL: Brown fine to corse Sand, trace Silt, trace fine to coarse Gravel, trace Asphaltic concrete fragments, loose-moist		6					
					ALLUVIUM: Gray Brown Silty fine Sand, loose-damp to moist		11					
	X	2			Dark Gray Brown fine Sandy Silt, little Clay, very loose-very moist		35					
5												
	X	4			Gray Brown Silty fine Sand to fine Sandy Silt, trace Clay, trace Iron oxide staining, loose-moist to very moist		27					
	X	9					8					
10												
					Brown Silty fine to medium Sand, trace coarse Sand, trace Iron oxide staining, medium dense-damp							
	X	15					3					
15												
					Boring Terminated at 15'							

TBL 15G227.GPJ SOCALGEO.GDT 2/3/16

TEST BORING LOG **PLATE B-14**

SOUTHERN CALIFORNIA GEOTECHNICAL

A California Corporation

BORING NO. B-15

JOB NO.: 15G227
PROJECT: Proposed C/I Development
LOCATION: El Monte, California

DRILLING DATE: 12/21/15
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Matt Manni

WATER DEPTH: Dry
CAVE DEPTH: 11.5 feet
READING TAKEN: At Completion

FIELD RESULTS					DESCRIPTION	LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	SURFACE ELEVATION: --- MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
					3± inches Asphaltic concrete, 3± inches Aggregate base							
		17			FILL: Brown fine to medium Sand, trace coarse Sand, trace fine to coarse Gravel, trace to little Silt, medium dense-damp	110	5					EI = 15 @ 0 to 5'
		8			ALLUVIUM: Gray Brown fine Sandy Silt, little Clay, trace Iron oxide staining, slightly porous, trace fine root fibers, loose-moist to very moist	81	14					
5		7				69	29					
		6				90	22					
		8				98	17					
10												
					Gray Brown Silty fine Sand, trace medium to coarse Sand, some Iron oxide staining, medium dense-moist to very moist							
		12					12					
15												
		16					17					
20					Boring Terminated at 20'							

TBL 15G227.GPJ SOCALGEO.GDT 2/3/16

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-16

JOB NO.: 15G227
PROJECT: Proposed C/I Development
LOCATION: El Monte, California

DRILLING DATE: 12/21/15
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Matt Manni

WATER DEPTH: Dry
CAVE DEPTH: 6 feet
READING TAKEN: At Completion

SURFACE ELEVATION: --- MSL

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
					2± inches Asphaltic concrete, no discernible Aggregate base							
	X	11			FILL: Dark Gray Brown Silty fine to medium Sand, trace coarse Sand, trace fine to coarse Gravel, trace Clay, mottled, medium dense-moist to very moist		12					
	X	8			@ 3½ to 5 feet, loose		14					
5												
	X	13			ALLUVIUM: Brown Silty fine Sand, trace medium to coarse Sand, trace Iron oxide staining, loose to medium dense-moist to very moist		9					
	X	10					12					
10												
					Light Gray Brown fine Sand, little Silt, trace Iron oxide staining, medium dense-damp to moist							
	X	17					7					
15												
					Boring Terminated at 15'							

TBL 15G227.GPJ SOCALGEO.GDT 2/3/16

BORING NO. B-17

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

JOB NO.: 15G227	DRILLING DATE: 12/21/15	WATER DEPTH: Dry
PROJECT: Proposed C/I Development	DRILLING METHOD: Hollow Stem Auger	CAVE DEPTH: 37 feet
LOCATION: El Monte, California	LOGGED BY: Matt Manni	READING TAKEN: At Completion

DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: --- MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
					4± inches Asphaltic concrete, no discernible Aggregate base							
	X	14			FILL: Brown Silty fine to medium Sand, trace coarse Sand, trace fine Gravel, mottled, trace fine root fibers, medium dense-damp		6					
	X	15					6					
5												
	X	11			ALLUVIUM: Brown Silty fine Sand, trace medium to coarse Sand, trace calcareous veining, slightly porous, medium dense-moist		9					
	X	16			Light Gray Brown fine to coarse Sand, trace fine to coarse Gravel, trace Silt, medium dense-dry to damp		2			4		
10												
	X	32			@ 13½ to 20 feet, little fine to coarse Gravel, occasional Cobbles, dense		2					
15												
	X	48					3					
20												
					Gray Brown Silty fine Sand, trace medium to coarse Sand, trace Iron oxide staining, dense-moist to very moist							
	X	31					14					
25												
					Gray Brown fine to coarse Sand, trace fine Gravel, trace Silt, very dense-damp							
	X	60					6					
30												
	X	57			@ 33½ to 35 feet, trace fine to coarse Gravel		4					

TBL 15G227.GPJ SOCALGEO.GDT 2/3/16

BORING NO. B-17

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

JOB NO.: 15G227 | DRILLING DATE: 12/21/15 | WATER DEPTH: Dry
PROJECT: Proposed C/I Development | DRILLING METHOD: Hollow Stem Auger | CAVE DEPTH: 37 feet
LOCATION: El Monte, California | LOGGED BY: Matt Manni | READING TAKEN: At Completion

DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION (Continued)	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
					Gray Brown fine to coarse Sand, trace fine Gravel, trace Silt, very dense-damp							
	X	46			@ 38½ to 40 feet, dense		3					
40												
					Gray Brown fine to medium Sand, trace to little coarse Sand, trace fine Gravel, trace Silt, dense-damp							
	X	42					5					
45												
	X	39					3					
50												
					Boring Terminated at 50'							

FIELD RESULTS / LABORATORY RESULTS

TBL 15G227.GPJ SOCALGEO.GDT 2/3/16

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-18

JOB NO.: 15G227
PROJECT: Proposed C/I Development
LOCATION: El Monte, California

DRILLING DATE: 1/6/16
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Matt Manni

WATER DEPTH: Dry
CAVE DEPTH: 3 feet
READING TAKEN: At Completion

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: --- MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
					3± inches Asphaltic concrete, 3± inches Aggregate base							
	X	5			Light Brown Clayey fine Sand to fine Sandy Clay, trace Iron oxide staining, loose to soft-very moist		29					
	X	7			ALLUVIUM: Dark Brown Silty Clay to Clayey Silt, trace to little fine Sand, trace Iron oxide staining, trace calcareous veining, slightly porous, loose-very moist		36					
5					Boring Terminated at 5'							

TBL 15G227.GPJ SOCALGEO.GDT 2/3/16

TEST BORING LOG

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-19

JOB NO.: 15G227
PROJECT: Proposed C/I Development
LOCATION: El Monte, California

DRILLING DATE: 1/6/16
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Matt Manni

WATER DEPTH: Dry
CAVE DEPTH: 3 feet
READING TAKEN: At Completion

DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: --- MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	COMMENTS
					2± inches Gravel							
	X	16			FILL: Brown Silty fine to medium Sand, trace coarse Sand, trace fine Gravel, trace Asphaltic concrete fragments, medium dense-damp to moist		7					
	X	14			POSSIBLE FILL: Light Gray Brown fine to coarse Sand, trace Silt, medium dense-damp		3					
5												
					Boring Terminated at 5'							

TBL 15G227.GPJ SOCALGEO.GDT 2/3/16

TEST BORING LOG

PLATE B-19

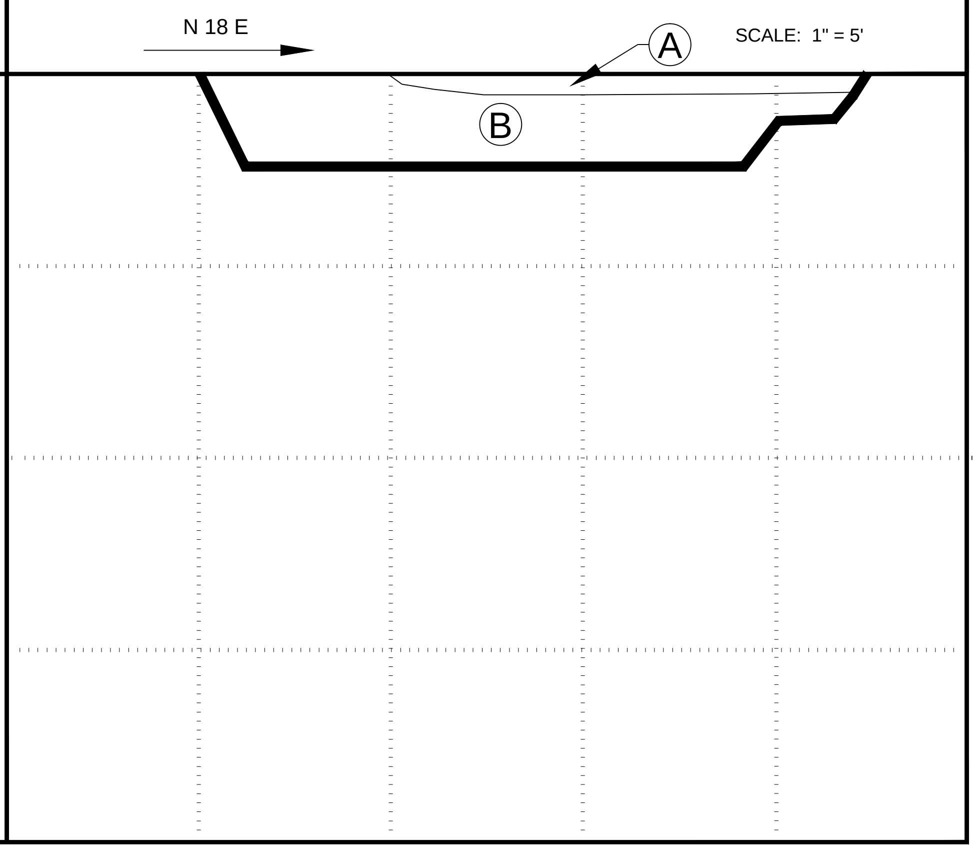
SOUTHERN CALIFORNIA GEOTECHNICAL

TRENCH NO.
T-1

JOB NO.: 15G227	EQUIPMENT USED: Backhoe	WATER DEPTH: Dry
PROJECT: Proposed C/I Development	LOGGED BY: Daryl Kas	SEEPAGE DEPTH: Dry
LOCATION: El Monte, CA	ORIENTATION: N 18 E	READINGS TAKEN: At Completion
DATE: 1-5-2016	TOP OF TRENCH ELEVATION:	

DEPTH	SAMPLE	DRY DENSITY (PCF)	MOISTURE (%)	EARTH MATERIALS DESCRIPTION
				A: PAVEMENT: Crushed Asphalt, 6" thick B: FILL: Gray Brown Silty fine Sand, trace medium to coarse Sand, trace fine to coarse Gravel, trace debris (Asphalt, Concrete, Plastic fragments), medium dense - damp to moist
				Trench Terminated @ 2.5 feet
5				
10				
15				

GRAPHIC REPRESENTATION



KEY TO SAMPLE TYPES:
B - BULK SAMPLE (DISTURBED)
R - RING SAMPLE 2-1/2" DIAMETER
(RELATIVELY UNDISTURBED)

TRENCH LOG

PLATE B-20

SOUTHERN CALIFORNIA GEOTECHNICAL

**TRENCH NO.
T-2**

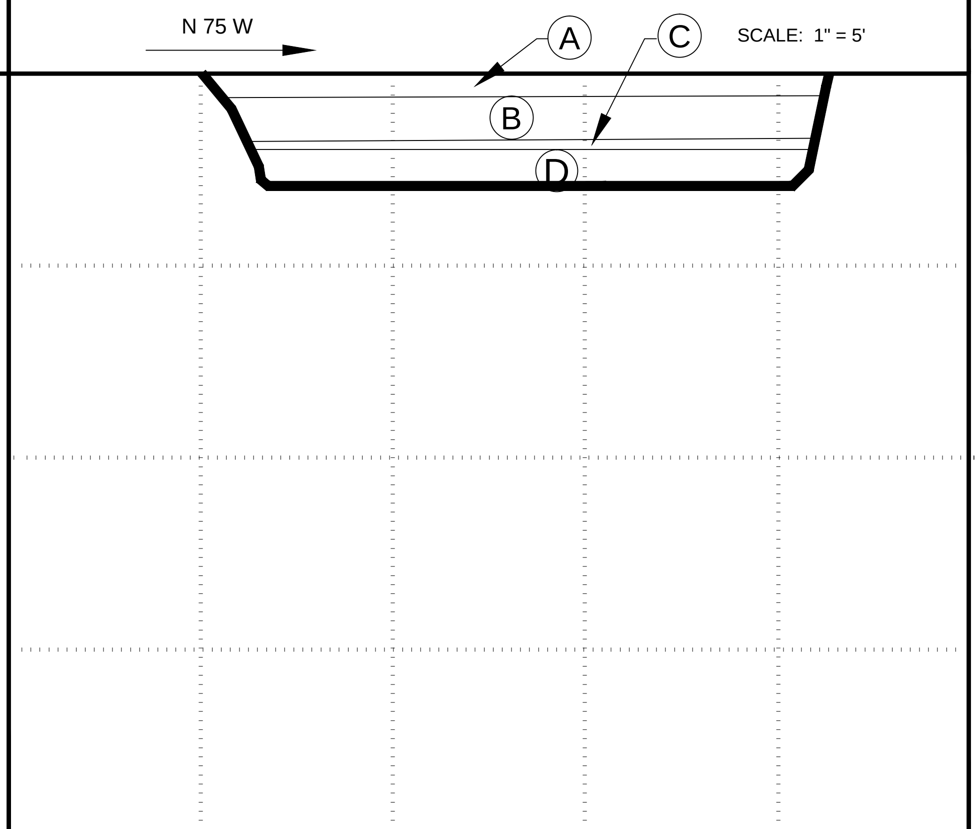
JOB NO.: 15G227-1
PROJECT: Proposed C/I Development
LOCATION: El Monte, CA
DATE: 1-5-2016

EQUIPMENT USED: Backhoe
LOGGED BY: Daryl Kas
ORIENTATION: N 75 W
TOP OF TRENCH ELEVATION:

WATER DEPTH: Dry
SEEPAGE DEPTH: Dry
READINGS TAKEN: At Completion

DEPTH	SAMPLE	DRY DENSITY (PCF)	MOISTURE (%)	EARTH MATERIALS DESCRIPTION
				A: PAVEMENTS: Crushed Asphalt, 6" thick B: FILL: Dark Gray Brown Silty fine Sand, trace medium to coarse Sand, mottled, trace debris (Asphalt, Concrete, and Brick fragments) medium dense - damp C: PAVEMENTS: Asphalt - 4" thick D: FILL: Dark Gray Brown Silty fine Sand, trace medium to coarse Sand, occasional Cobbles, mottled, medium dense - damp Trench Terminated @ 3 feet
5				
10				
15				

GRAPHIC REPRESENTATION



KEY TO SAMPLE TYPES:
B - BULK SAMPLE (DISTURBED)
R - RING SAMPLE 2-1/2" DIAMETER
(RELATIVELY UNDISTURBED)

TRENCH LOG

PLATE B-21

SOUTHERN CALIFORNIA GEOTECHNICAL

TRENCH NO.
T-3

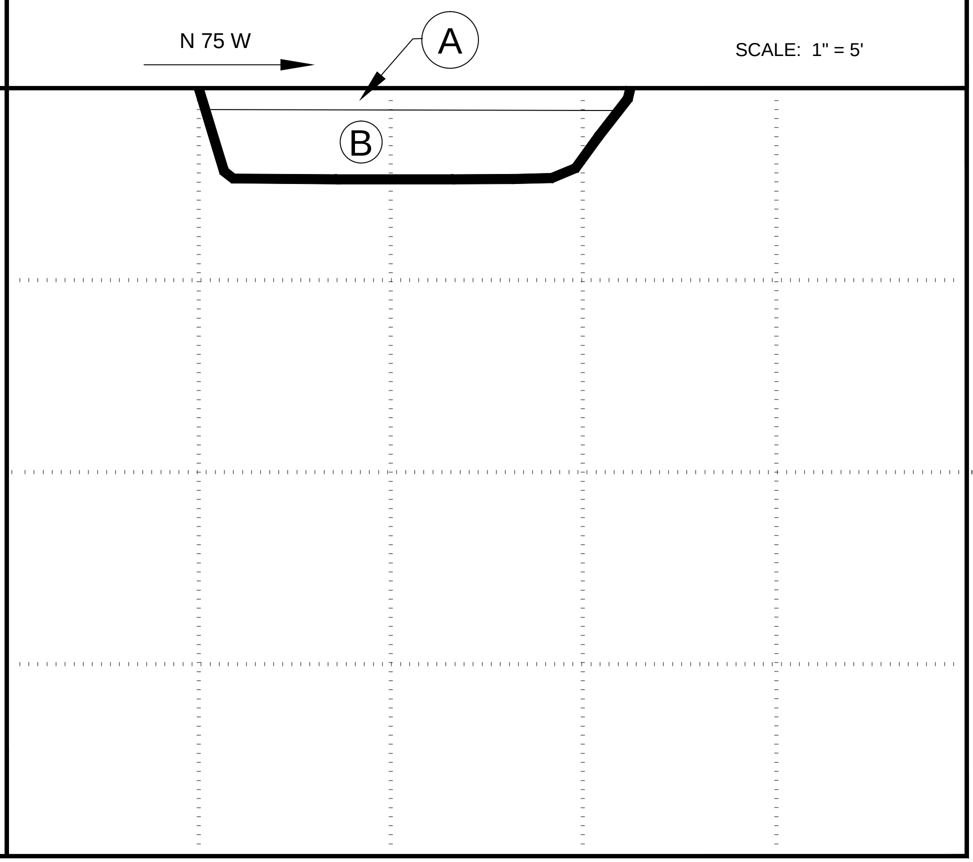
JOB NO.: 15G227-1
PROJECT: Proposed C/I Development
LOCATION: El Monte, CA
DATE: 1-5-2016

EQUIPMENT USED: Backhoe
LOGGED BY: Daryl Kas
ORIENTATION: N 75 W
TOP OF TRENCH ELEVATION:

WATER DEPTH: Dry
SEEPAGE DEPTH: Dry
READINGS TAKEN: At Completion

DEPTH	SAMPLE	DRY DENSITY (PCF)	MOISTURE (%)	EARTH MATERIALS DESCRIPTION
				A: PAVEMENTS: Crushed Asphalt, 6" thick B: FILL: Gray Brown Silty fine to medium Sand, trace coarse Sand, trace fine to coarse Gravel, trace debris (Wire, Asphalt fragments) medium dense - damp
				Trench Terminated @ 2.5 feet
5				
10				
15				

GRAPHIC REPRESENTATION



KEY TO SAMPLE TYPES:
B - BULK SAMPLE (DISTURBED)
R - RING SAMPLE 2-1/2" DIAMETER
(RELATIVELY UNDISTURBED)

TRENCH LOG

PLATE B-22

SOUTHERN CALIFORNIA GEOTECHNICAL

TRENCH NO.
T-4

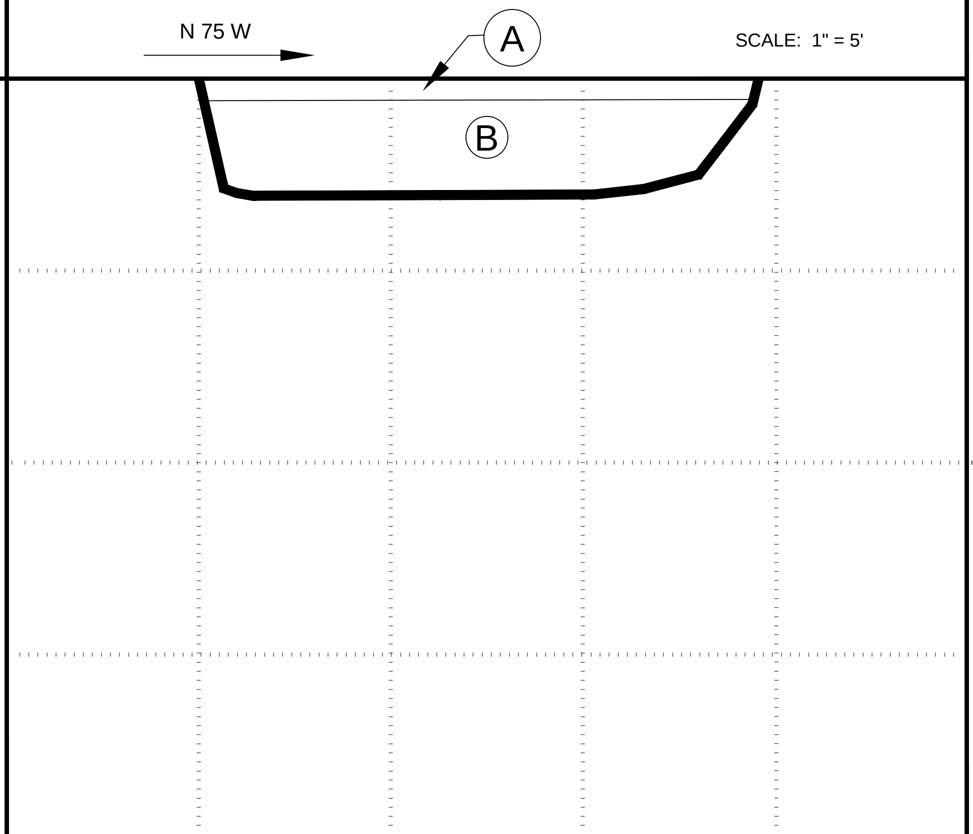
JOB NO.: 15G227
PROJECT: Proposed C/I Development
LOCATION: El Monte, CA
DATE: 1-5-2016

EQUIPMENT USED: Backhoe
LOGGED BY: Daryl Kas
ORIENTATION: N 75 W
TOP OF TRENCH ELEVATION:

WATER DEPTH: Dry
SEEPAGE DEPTH: Dry
READINGS TAKEN: At Completion

DEPTH	SAMPLE	DRY DENSITY (PCF)	MOISTURE (%)	EARTH MATERIALS DESCRIPTION
				A: PAVEMENTS: Crushed Asphalt, 6" thick B: FILL: Gray Brown Silty fine Sand, trace to little medium to coarse Sand, trace debris (Concrete, Asphalt fragments) mottled, medium dense - damp
				Trench Terminated @ 3 feet
5				
10				
15				

GRAPHIC REPRESENTATION



KEY TO SAMPLE TYPES:
B - BULK SAMPLE (DISTURBED)
R - RING SAMPLE 2-1/2" DIAMETER
(RELATIVELY UNDISTURBED)

TRENCH LOG

PLATE B-23

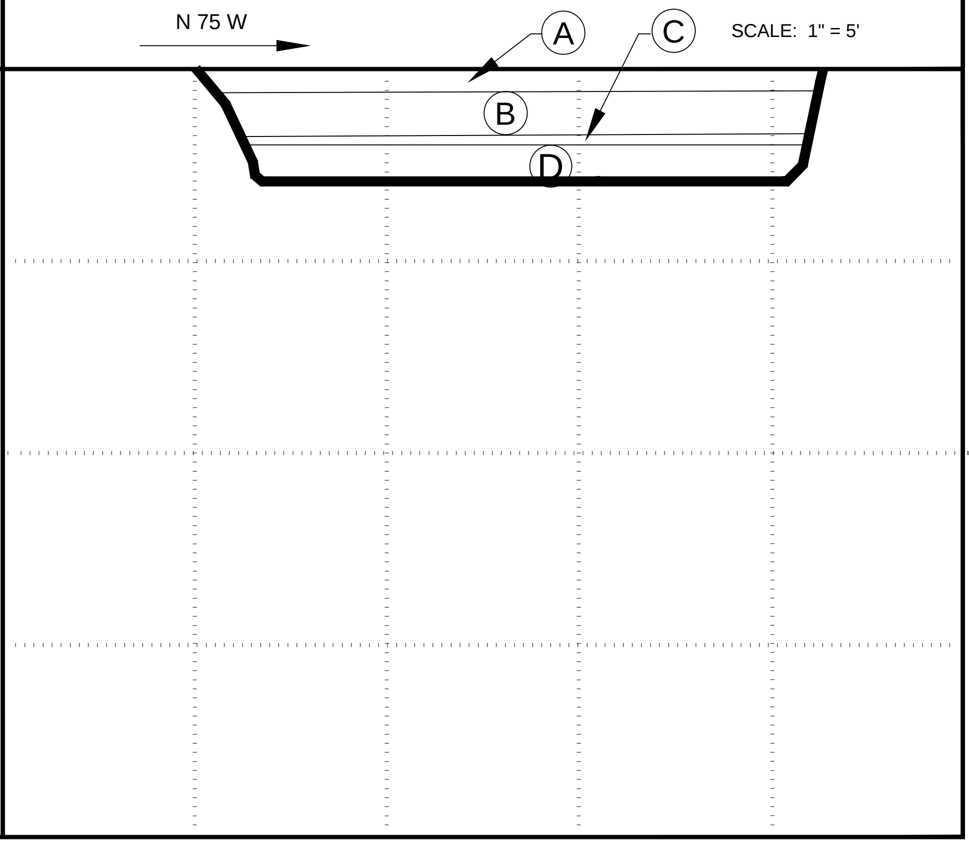
SOUTHERN CALIFORNIA GEOTECHNICAL

TRENCH NO. T-5

JOB NO.: 15G227-1	EQUIPMENT USED: Backhoe	WATER DEPTH: Dry
PROJECT: Proposed C/I Development	LOGGED BY: Daryl Kas	SEEPAGE DEPTH: Dry
LOCATION: El Monte, CA	ORIENTATION: N 75 W	READINGS TAKEN: At Completion
DATE: 1-5-2016	TOP OF TRENCH ELEVATION:	

DEPTH	SAMPLE	DRY DENSITY (PCF)	MOISTURE (%)	EARTH MATERIALS DESCRIPTION
				A: PAVEMENTS: Crushed Asphalt, 6" thick B: FILL: Dark Gray Brown Silty fine Sand, trace medium to coarse Sand, mottled, trace debris (Asphalt, Concrete, and Brick fragments) medium dense - damp C: PAVEMENTS: Asphalt - 4" thick D: FILL: Gray fine to coarse Sand, trace fine gravel, loose - dry to damp
				Trench Terminated @ 3 feet
5				
10				
15				

GRAPHIC REPRESENTATION

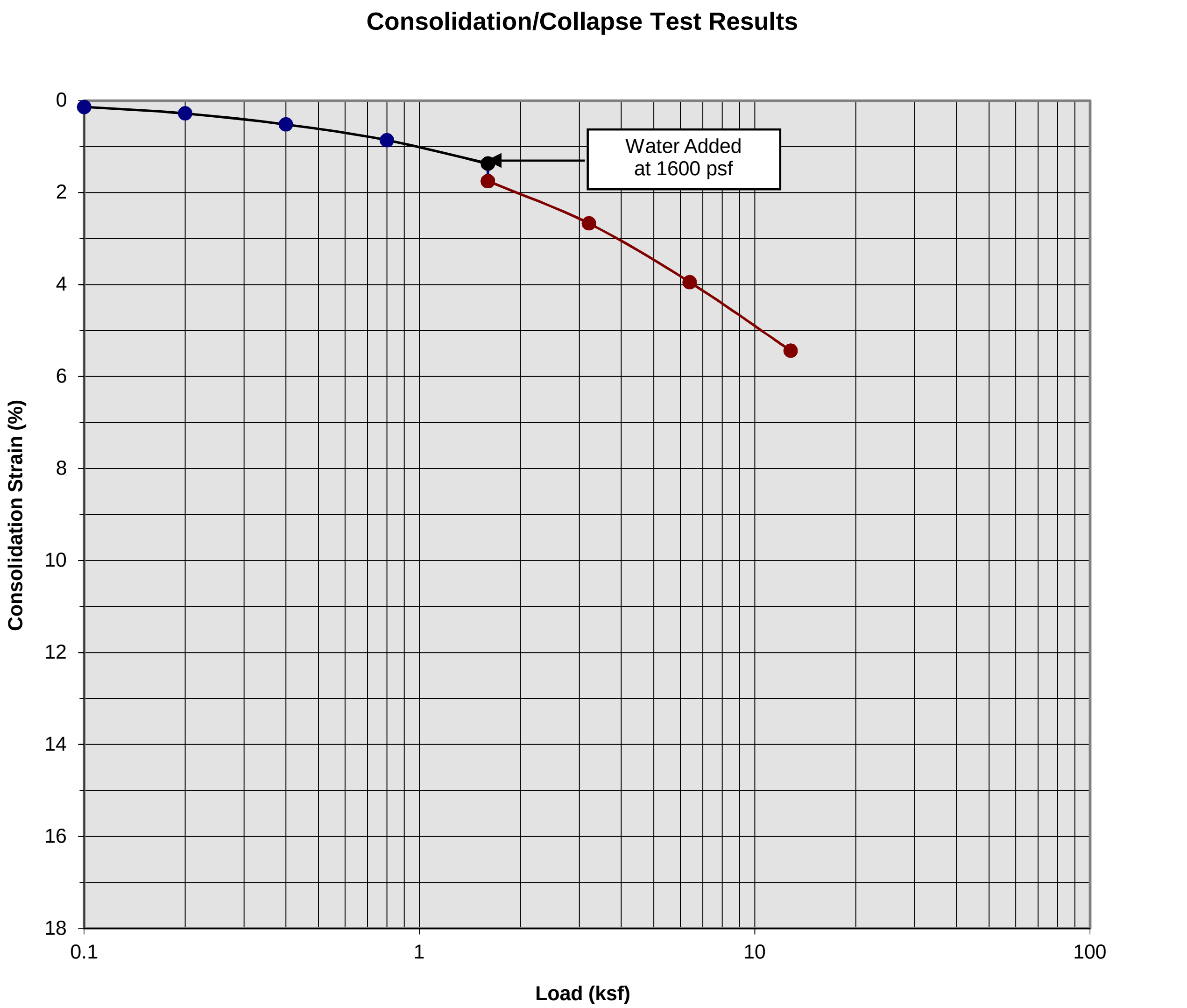


KEY TO SAMPLE TYPES:
B - BULK SAMPLE (DISTURBED)
R - RING SAMPLE 2-1/2" DIAMETER
(RELATIVELY UNDISTURBED)

TRENCH LOG

PLATE B-24

APPENDIX C

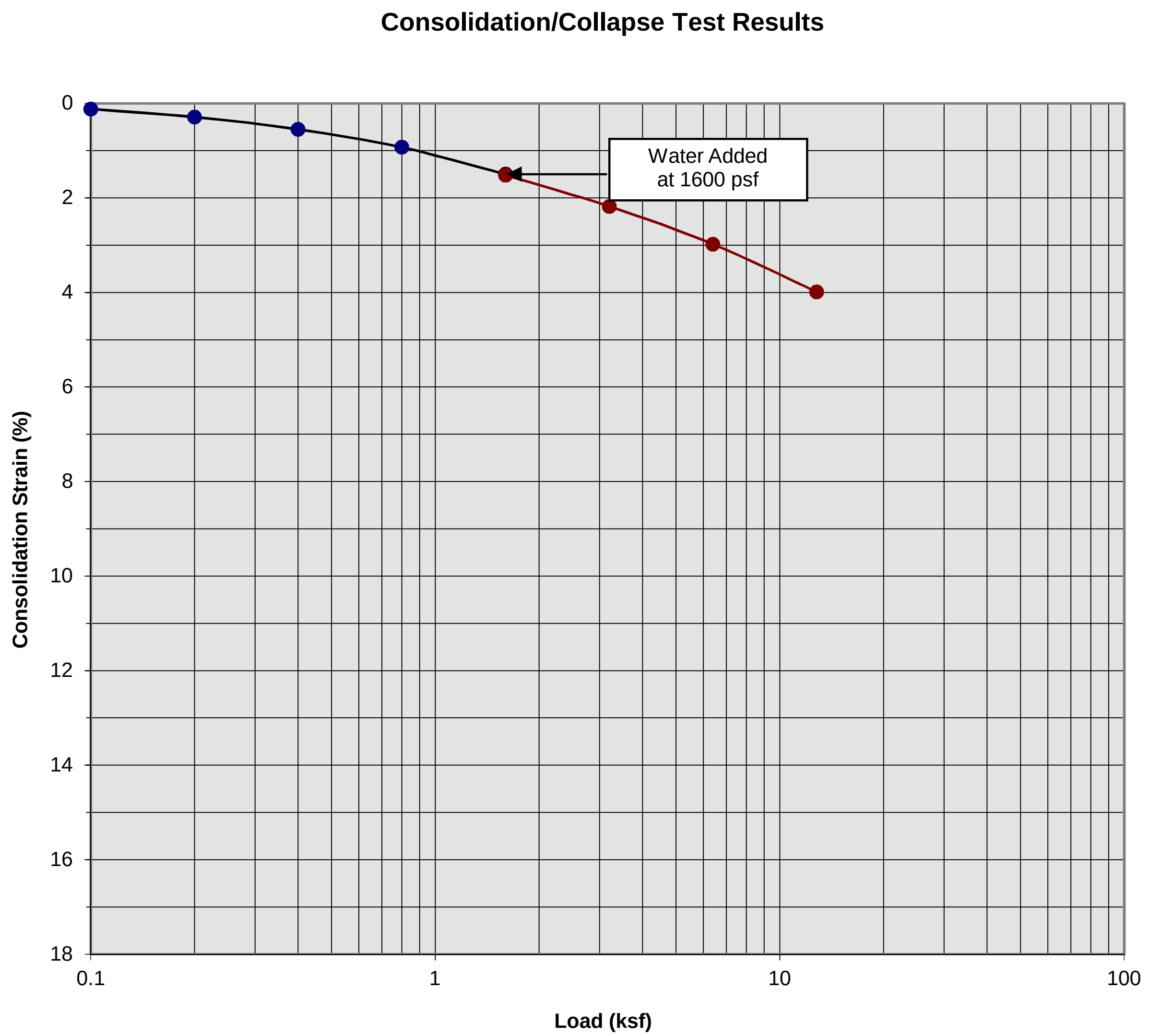


Classification: FILL: Brown Silty fine to medium Sand, trace coarse Sand

Boring Number:	B-2	Initial Moisture Content (%)	9
Sample Number:	---	Final Moisture Content (%)	13
Depth (ft)	1 to 2	Initial Dry Density (pcf)	114.8
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	121.0
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.38

Proposed Commercial/Industrial Development
El Monte, California
Project No. 15G227
PLATE C- 1

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation



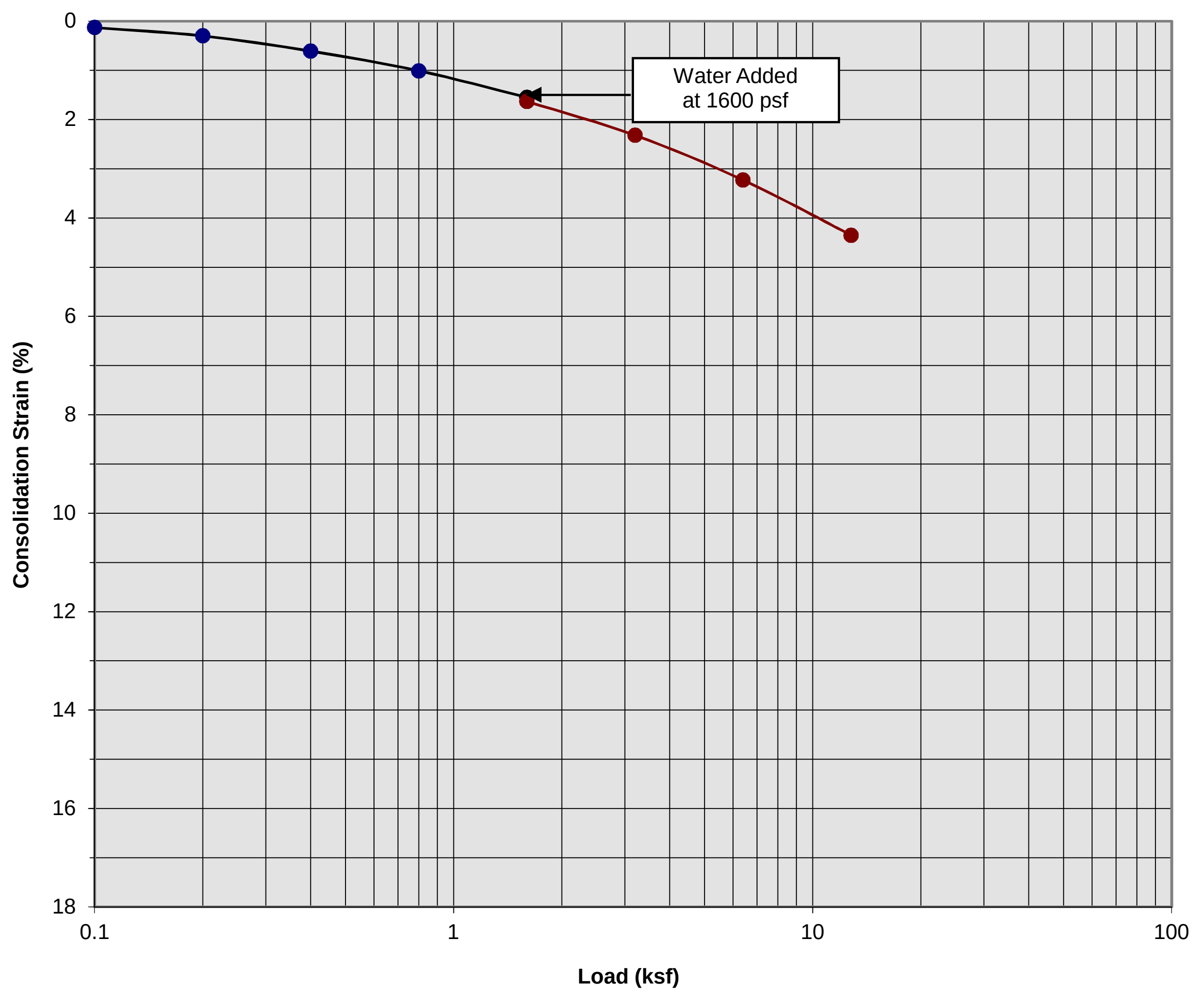
Classification: FILL: Brown Silty fine to medium Sand, trace coarse Sand

Boring Number:	B-2	Initial Moisture Content (%)	17
Sample Number:	---	Final Moisture Content (%)	18
Depth (ft)	3 to 4	Initial Dry Density (pcf)	112.7
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	117.4
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.02

Proposed Commercial/Industrial Development
El Monte, California
Project No. 15G227
PLATE C- 2

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

Consolidation/Collapse Test Results

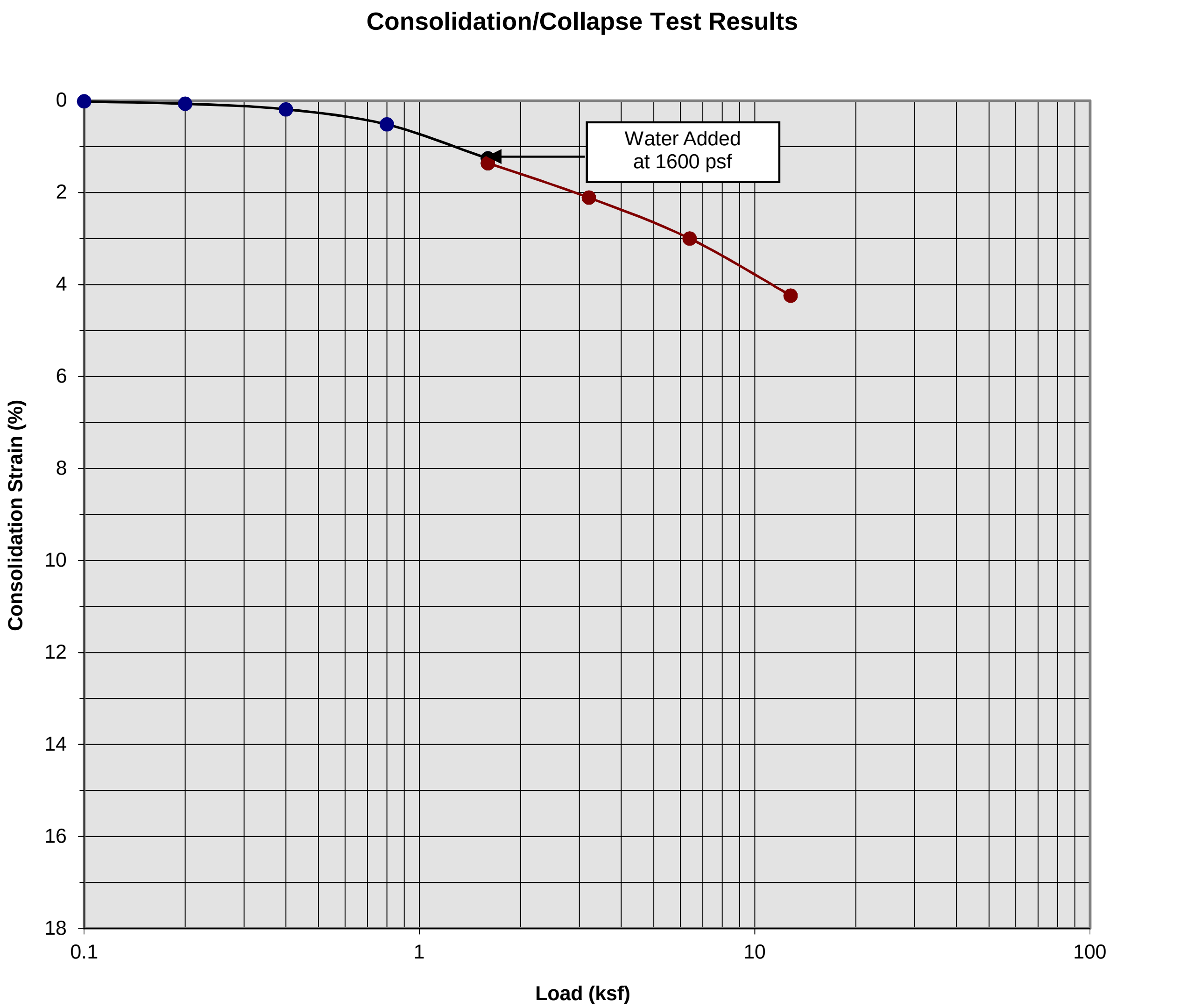


Classification: FILL: Dark Gray Silty fine to medium Sand, trace coarse Sand

Boring Number:	B-2	Initial Moisture Content (%)	11
Sample Number:	---	Final Moisture Content (%)	13
Depth (ft)	5 to 6	Initial Dry Density (pcf)	118.6
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	123.9
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.08

Proposed Commercial/Industrial Development
El Monte, California
Project No. 15G227
PLATE C- 3

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

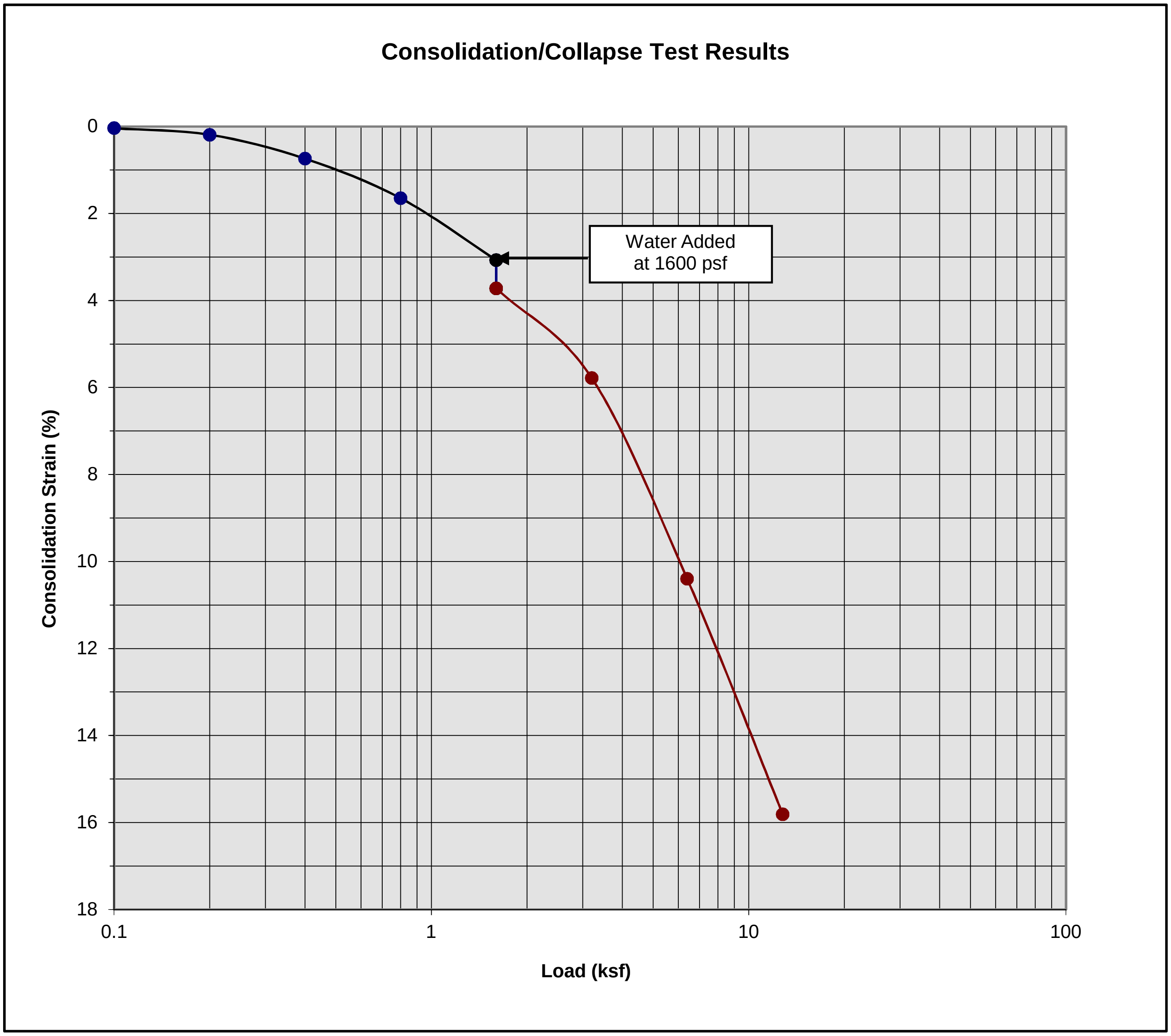


Classification: FILL: Dark Gray Silty fine to medium Sand, trace coarse Sand

Boring Number:	B-2	Initial Moisture Content (%)	12
Sample Number:	---	Final Moisture Content (%)	14
Depth (ft)	7 to 8	Initial Dry Density (pcf)	116.5
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	121.3
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.10

Proposed Commercial/Industrial Development
El Monte, California
Project No. 15G227
PLATE C- 4

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

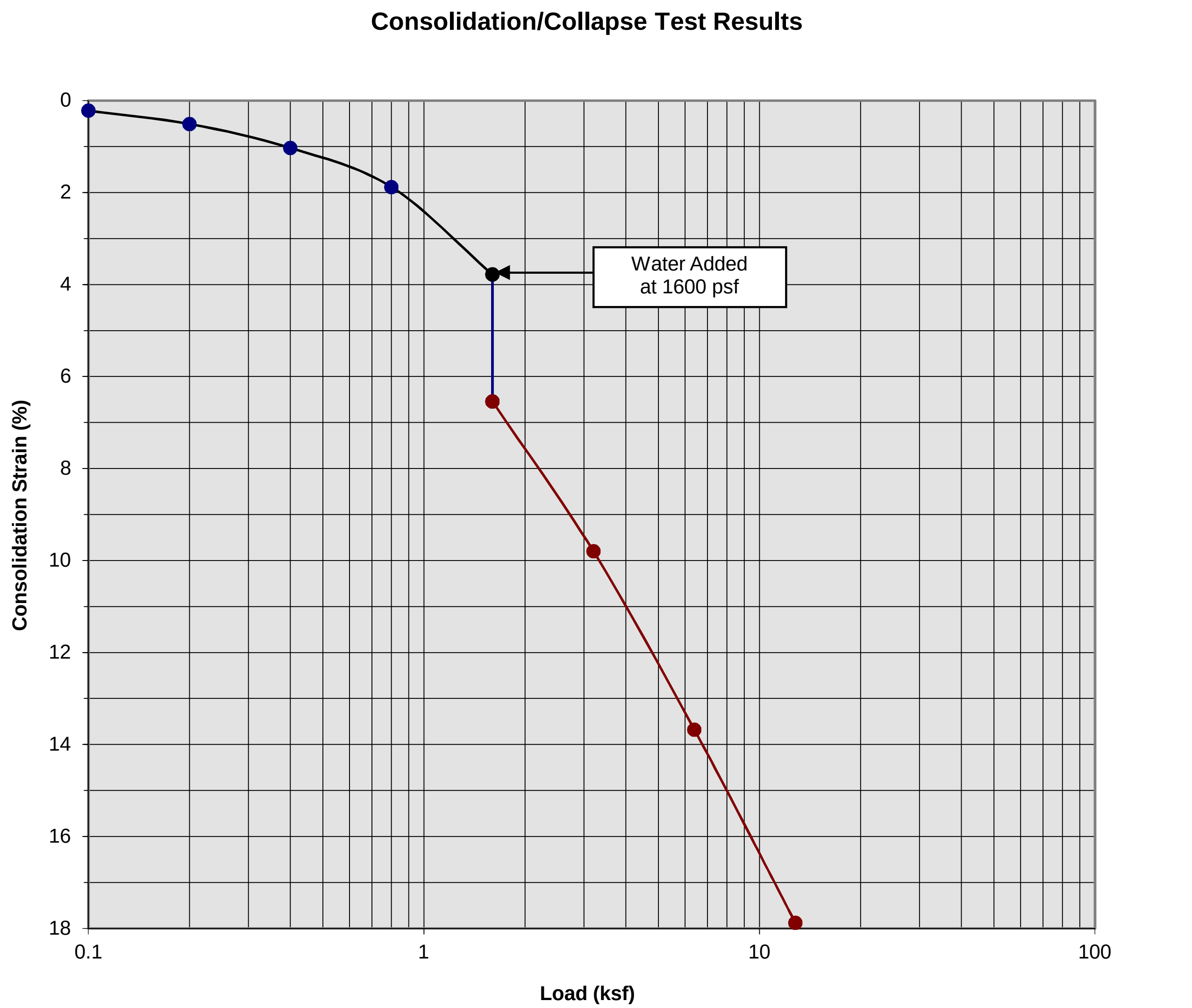


Classification: Light Gray Brown fine Sand, little Silt

Boring Number:	B-3	Initial Moisture Content (%)	15
Sample Number:	---	Final Moisture Content (%)	17
Depth (ft)	1 to 2	Initial Dry Density (pcf)	83.7
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	99.6
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.65

Proposed Commercial/Industrial Development
El Monte, California
Project No. 15G227
PLATE C- 5

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

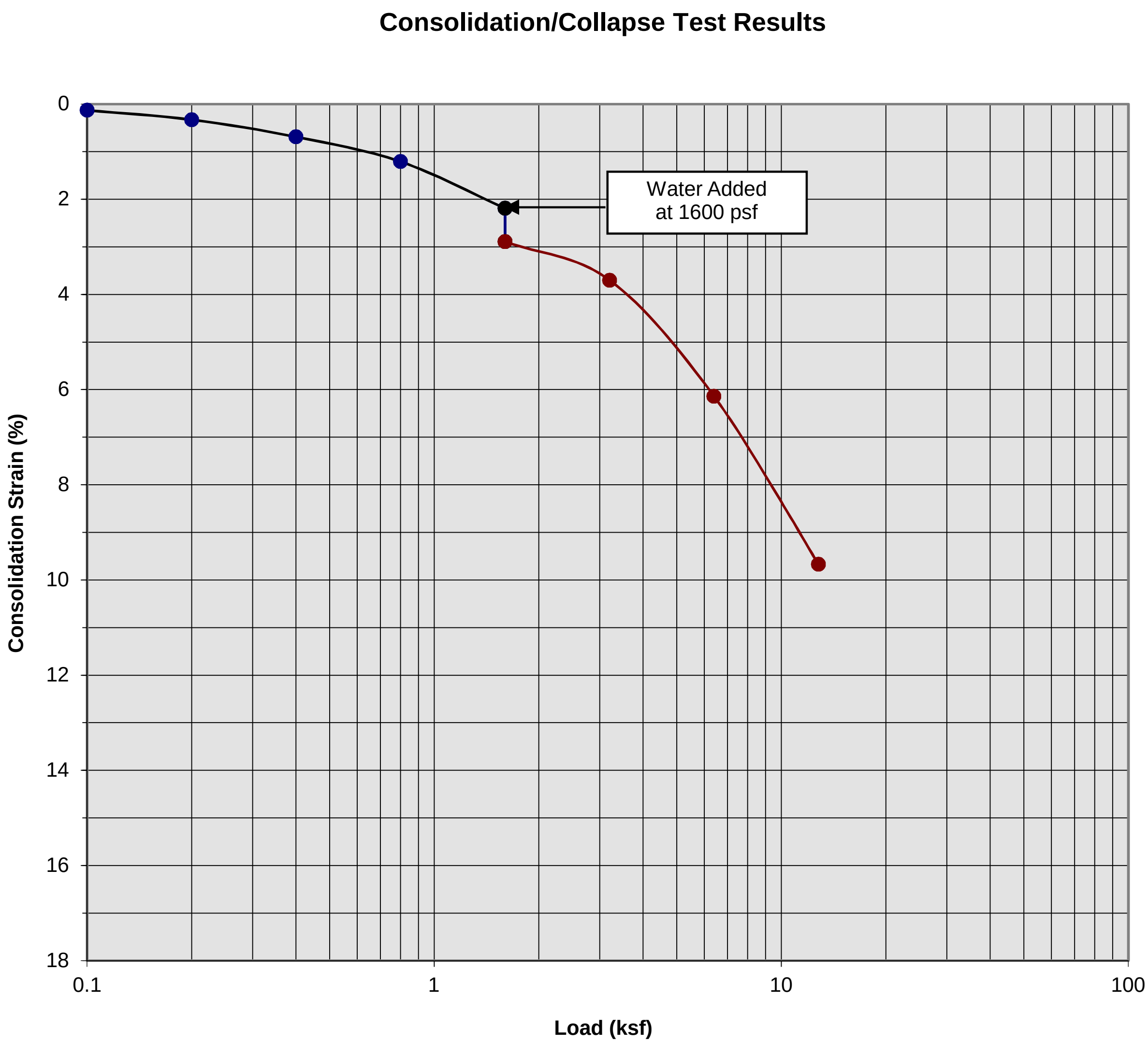


Classification: Dark Brown fine Sandy Clay

Boring Number:	B-3	Initial Moisture Content (%)	39
Sample Number:	---	Final Moisture Content (%)	38
Depth (ft)	3 to 4	Initial Dry Density (pcf)	70.8
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	86.2
Specimen Thickness (in)	1.0	Percent Collapse (%)	2.76

Proposed Commercial/Industrial Development
El Monte, California
Project No. 15G227
PLATE C- 6

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation



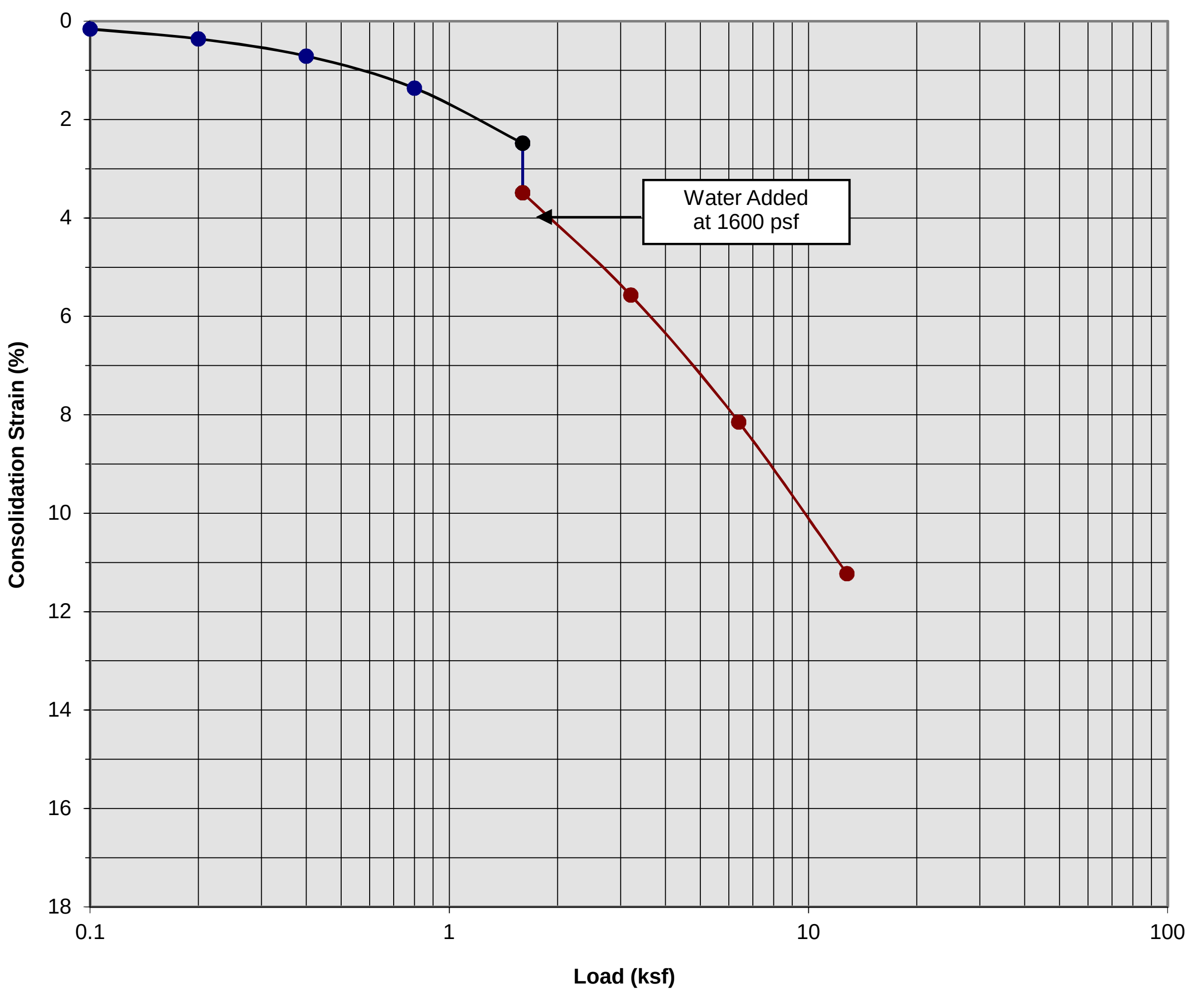
Classification: Dark Brown fine Sandy Clay

Boring Number:	B-3	Initial Moisture Content (%)	29
Sample Number:	---	Final Moisture Content (%)	31
Depth (ft)	5 to 6	Initial Dry Density (pcf)	90.2
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	96.3
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.70

Proposed Commercial/Industrial Development
El Monte, California
Project No. 15G227
PLATE C- 7

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

Consolidation/Collapse Test Results

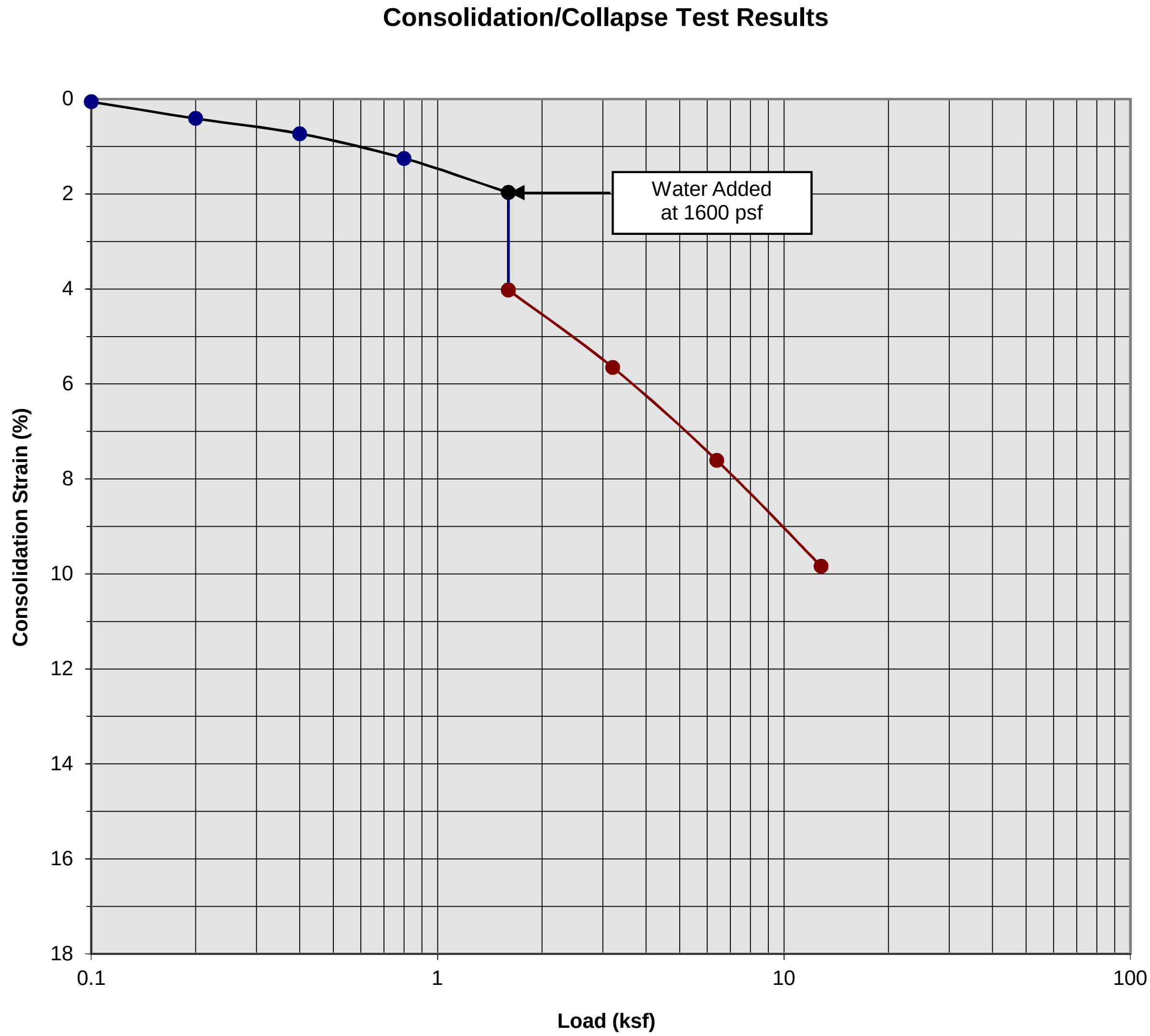


Classification: Light Brown fine Sandy Silt

Boring Number:	B-3	Initial Moisture Content (%)	18
Sample Number:	---	Final Moisture Content (%)	24
Depth (ft)	7 to 8	Initial Dry Density (pcf)	96.1
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	107.5
Specimen Thickness (in)	1.0	Percent Collapse (%)	1.01

Proposed Commercial/Industrial Development
El Monte, California
Project No. 15G227
PLATE C- 8

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

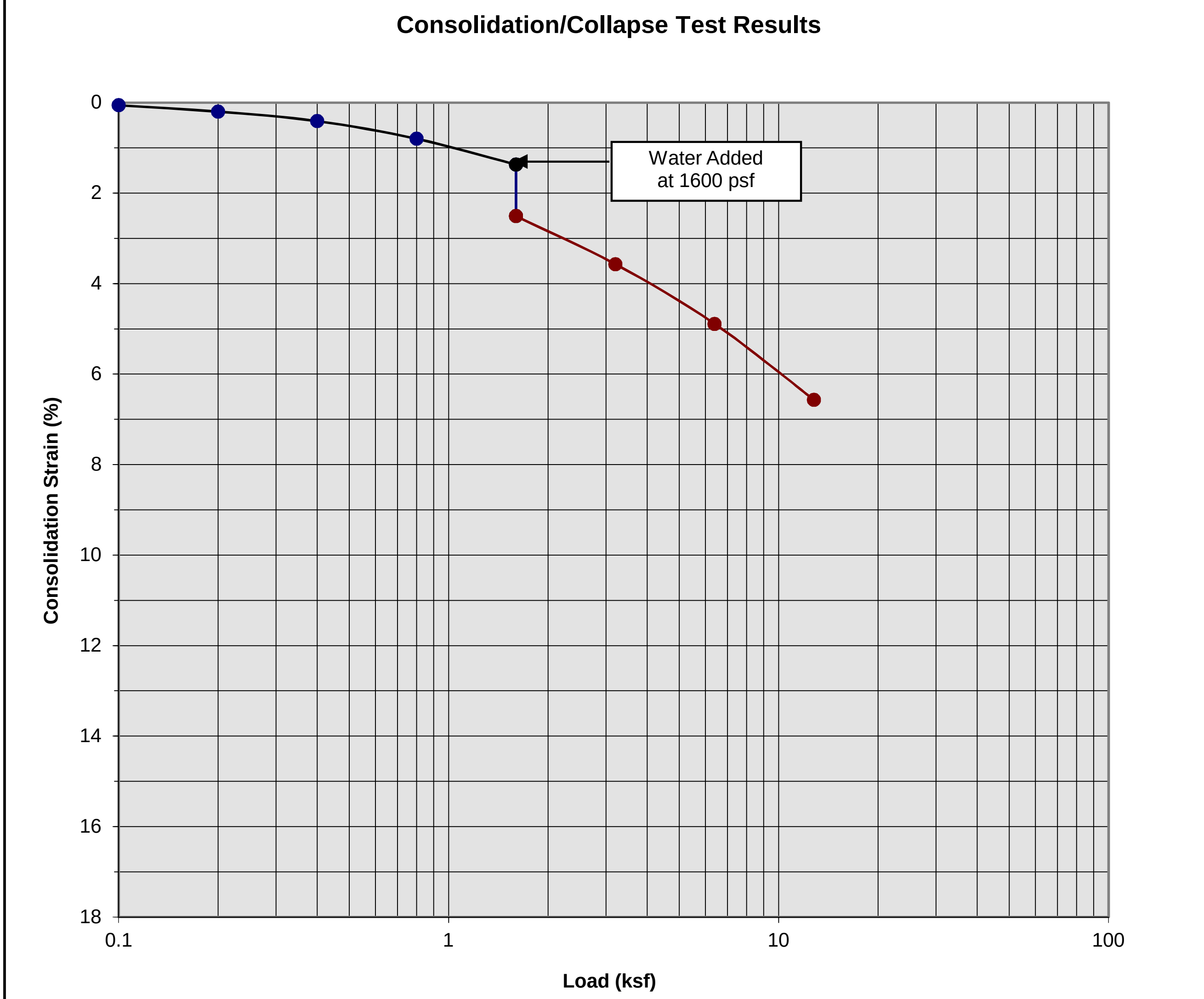


Classification: Gray Brown Silty fine Sand, trace medium to coarse Sand

Boring Number:	B-6	Initial Moisture Content (%)	8
Sample Number:	---	Final Moisture Content (%)	14
Depth (ft)	3 to 4	Initial Dry Density (pcf)	111.7
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	123.2
Specimen Thickness (in)	1.0	Percent Collapse (%)	2.05

Proposed Commercial/Industrial Development
El Monte, California
Project No. 15G227
PLATE C- 9

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

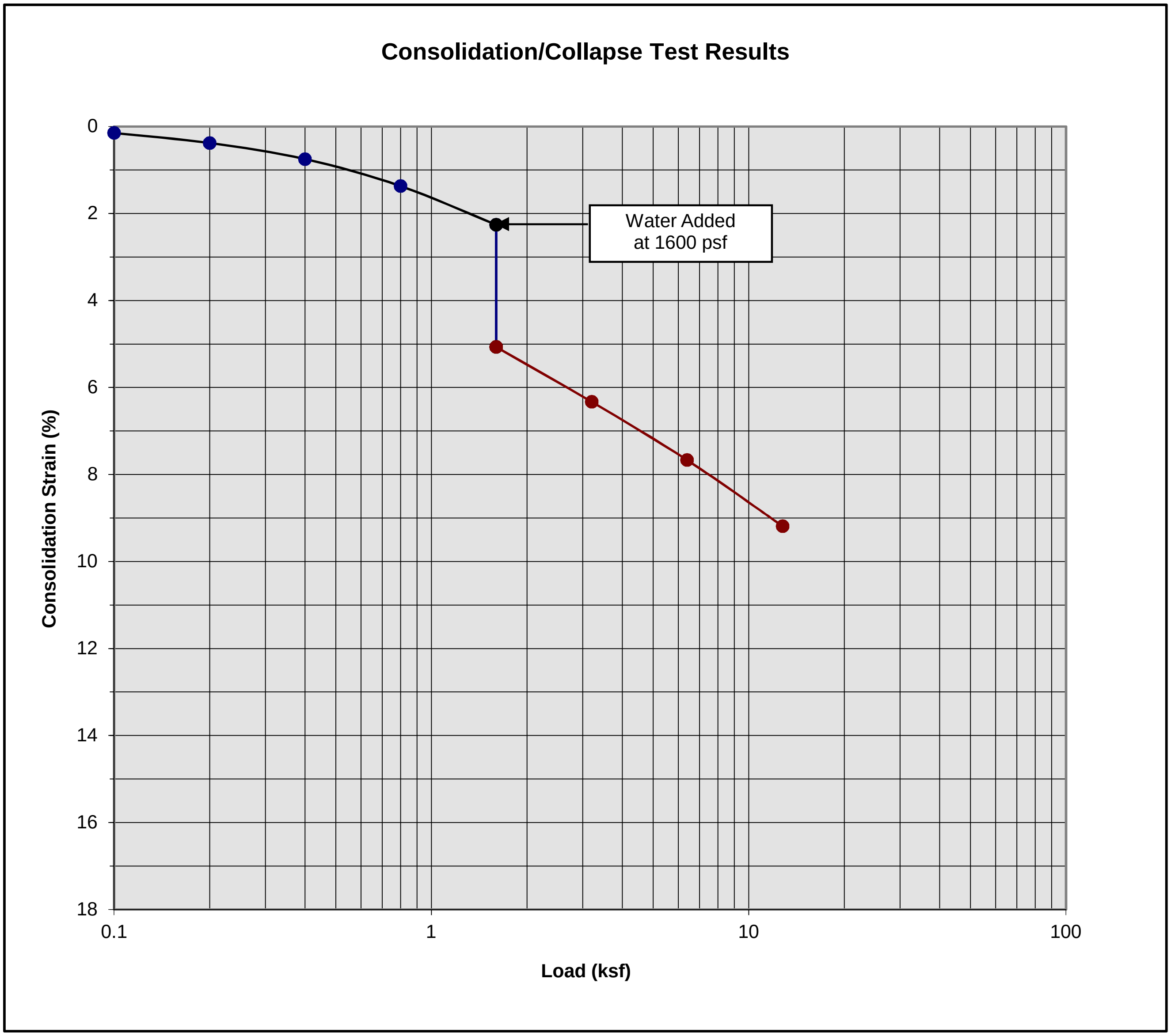


Classification: Gray Brown Silty fine Sand, trace medium to coarse Sand

Boring Number:	B-6	Initial Moisture Content (%)	8
Sample Number:	---	Final Moisture Content (%)	15
Depth (ft)	5 to 6	Initial Dry Density (pcf)	115.5
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	123.4
Specimen Thickness (in)	1.0	Percent Collapse (%)	1.14

Proposed Commercial/Industrial Development
El Monte, California
Project No. 15G227
PLATE C- 10

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

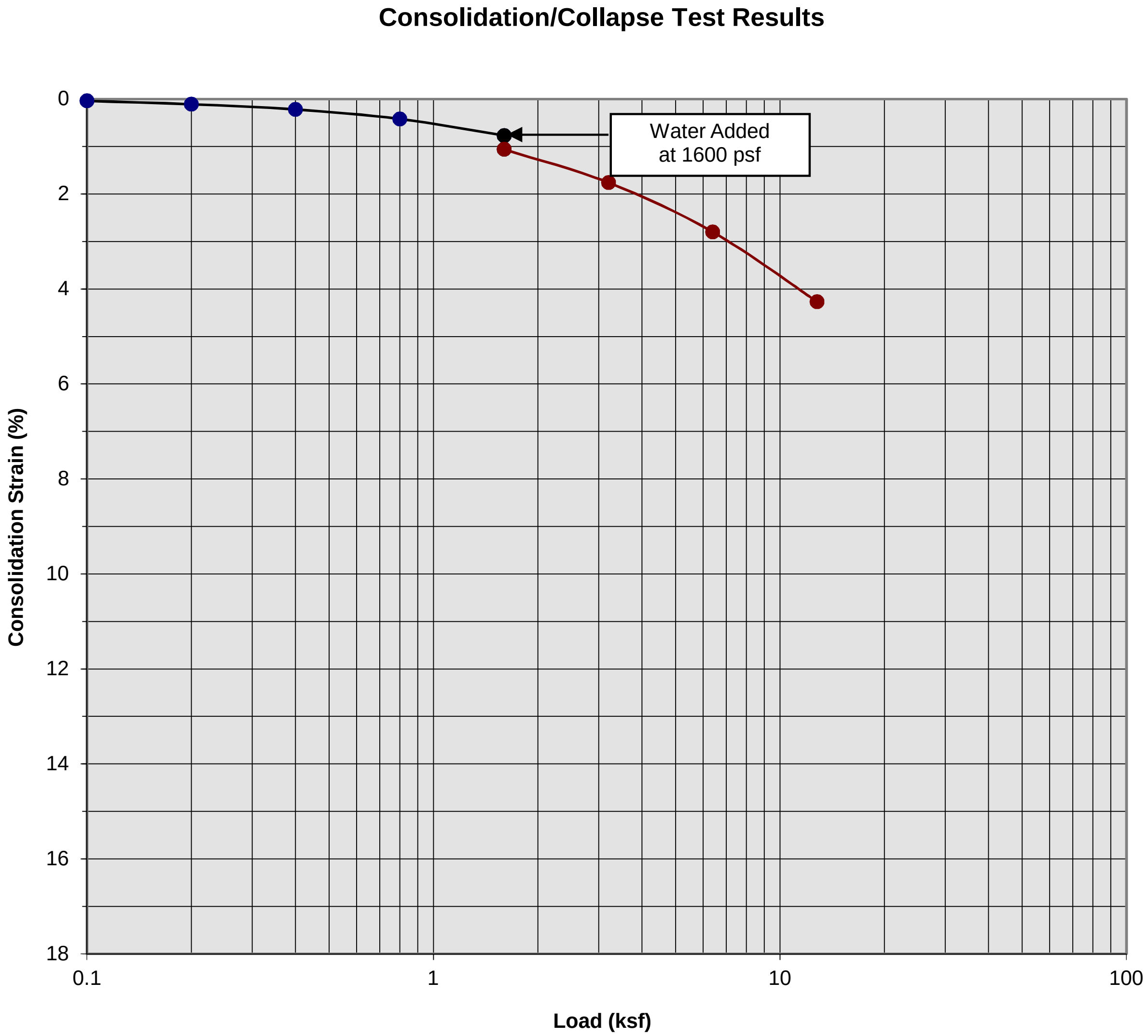


Classification: Brown fine Sand, little Silt, trace medium to coarse Sand

Boring Number:	B-6	Initial Moisture Content (%)	6
Sample Number:	---	Final Moisture Content (%)	14
Depth (ft)	7 to 8	Initial Dry Density (pcf)	107.0
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	117.2
Specimen Thickness (in)	1.0	Percent Collapse (%)	2.81

Proposed Commercial/Industrial Development
El Monte, California
Project No. 15G227
PLATE C- 11

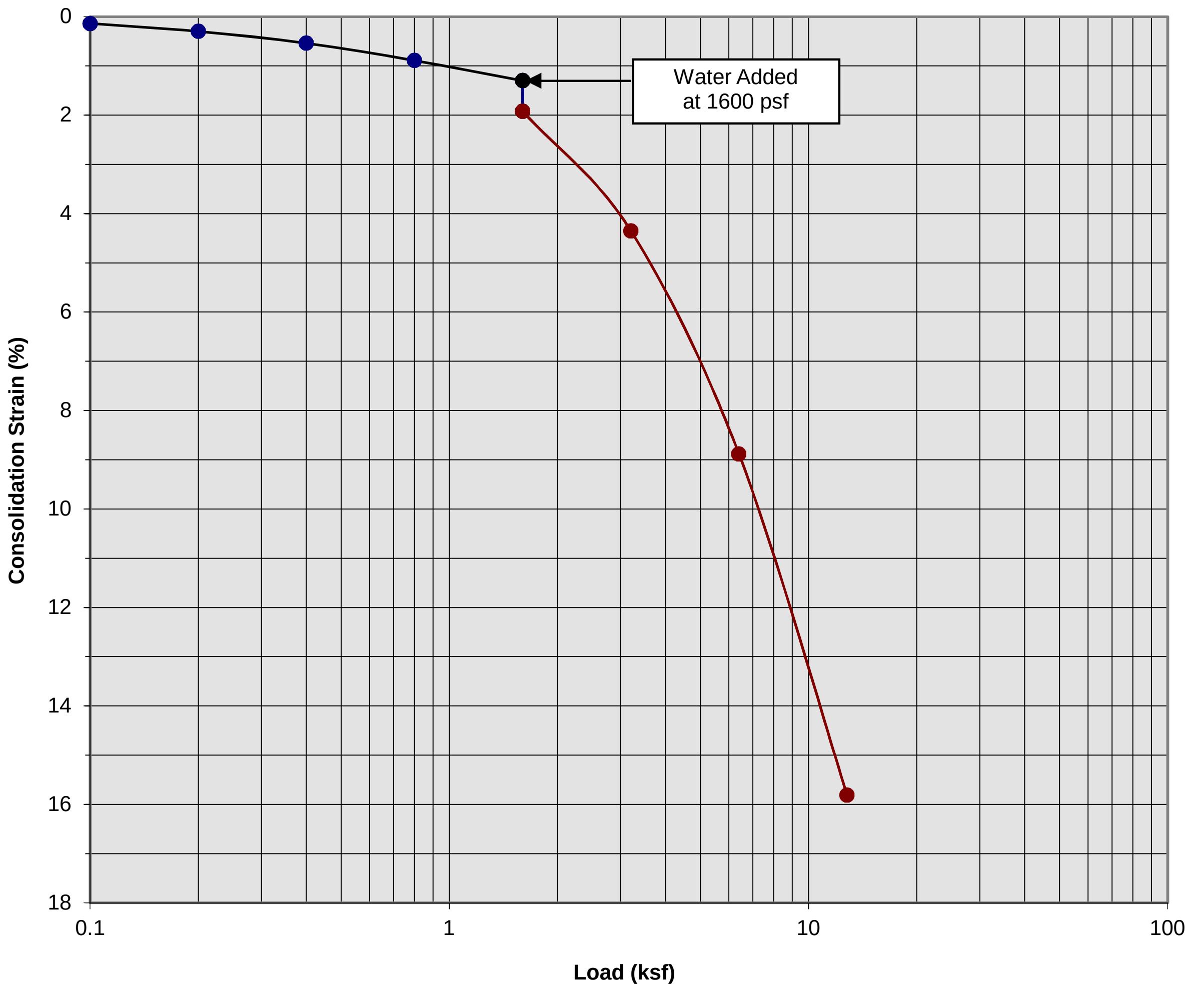
SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation



Classification: Brown fine Sand, little Silt, trace medium to coarse Sand

Boring Number:	B-6	Initial Moisture Content (%)	11
Sample Number:	---	Final Moisture Content (%)	13
Depth (ft)	9 to 10	Initial Dry Density (pcf)	113.6
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	123.6
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.29

Consolidation/Collapse Test Results



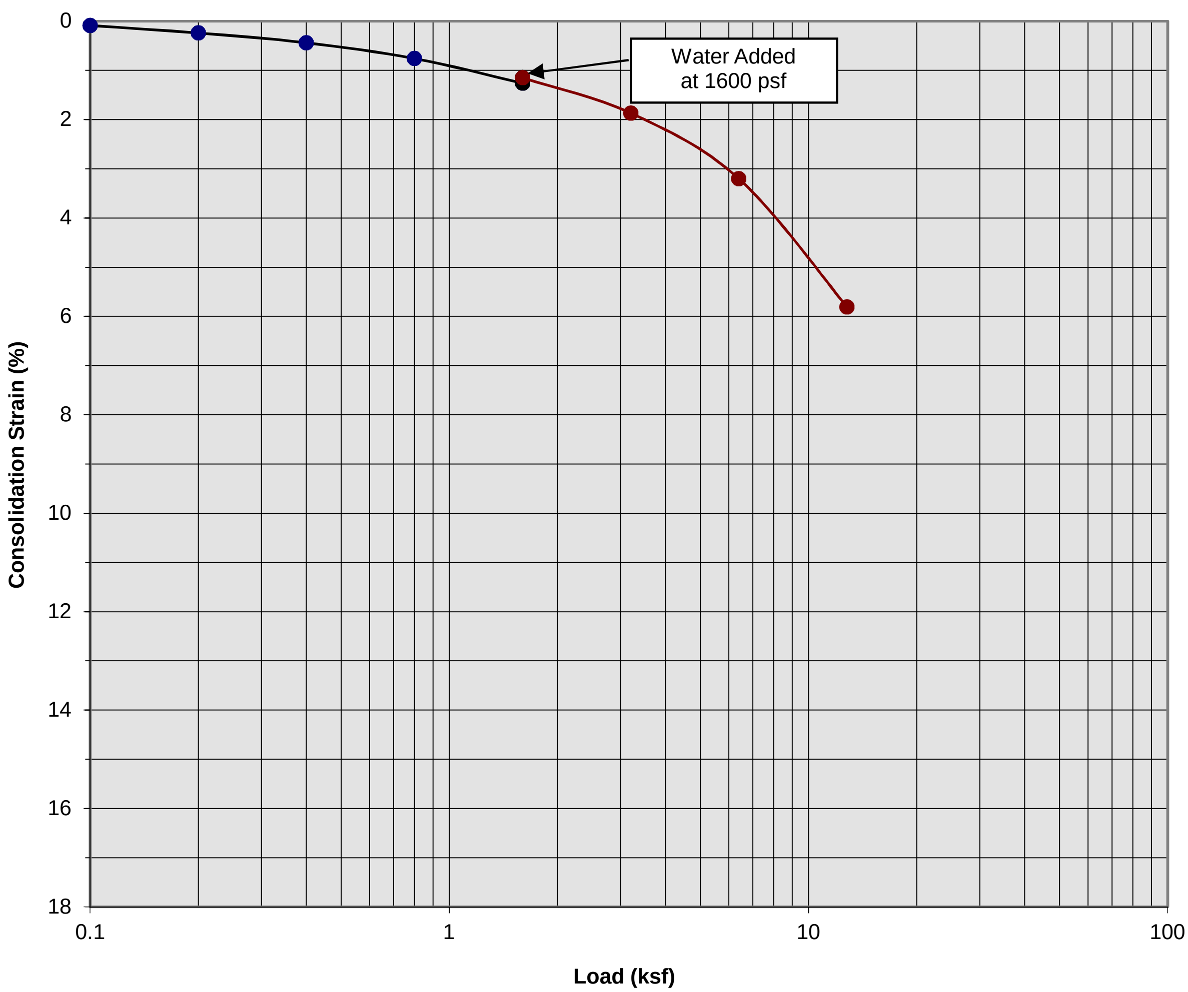
Classification: Dark Gray Brown Silty fine Sand to fine Sandy Silt

Boring Number:	B-13	Initial Moisture Content (%)	29
Sample Number:	---	Final Moisture Content (%)	42
Depth (ft)	3 to 4	Initial Dry Density (pcf)	61.7
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	77.2
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.62

Proposed Commercial/Industrial Development
El Monte, California
Project No. 15G227
PLATE C- 13

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

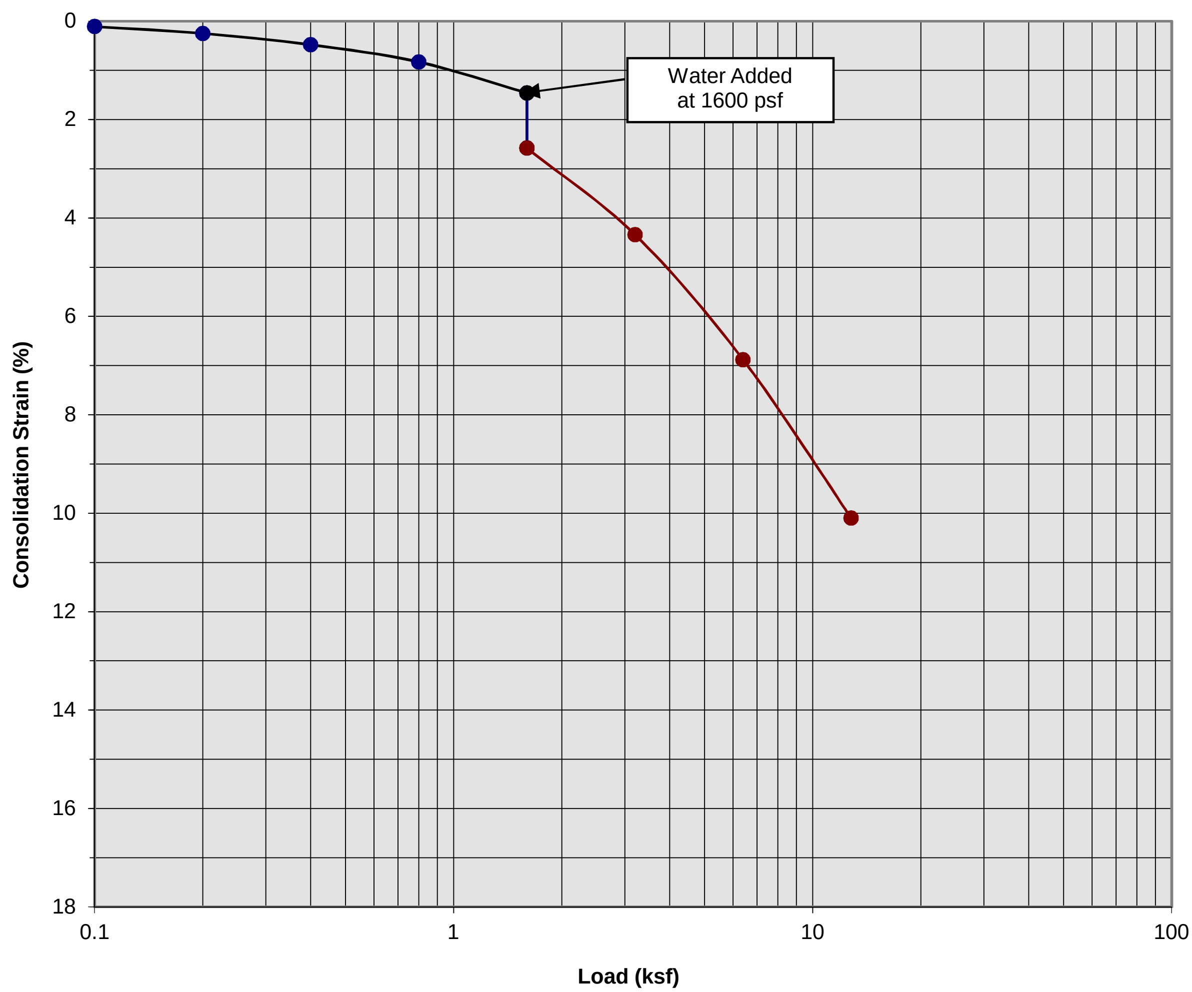
Consolidation/Collapse Test Results



Classification: Dark Gray Brown Silty fine Sand to fine Sandy Silt

Boring Number:	B-13	Initial Moisture Content (%)	30
Sample Number:	---	Final Moisture Content (%)	45
Depth (ft)	5 to 6	Initial Dry Density (pcf)	60.8
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	66.9
Specimen Thickness (in)	1.0	Percent Collapse (%)	-0.11

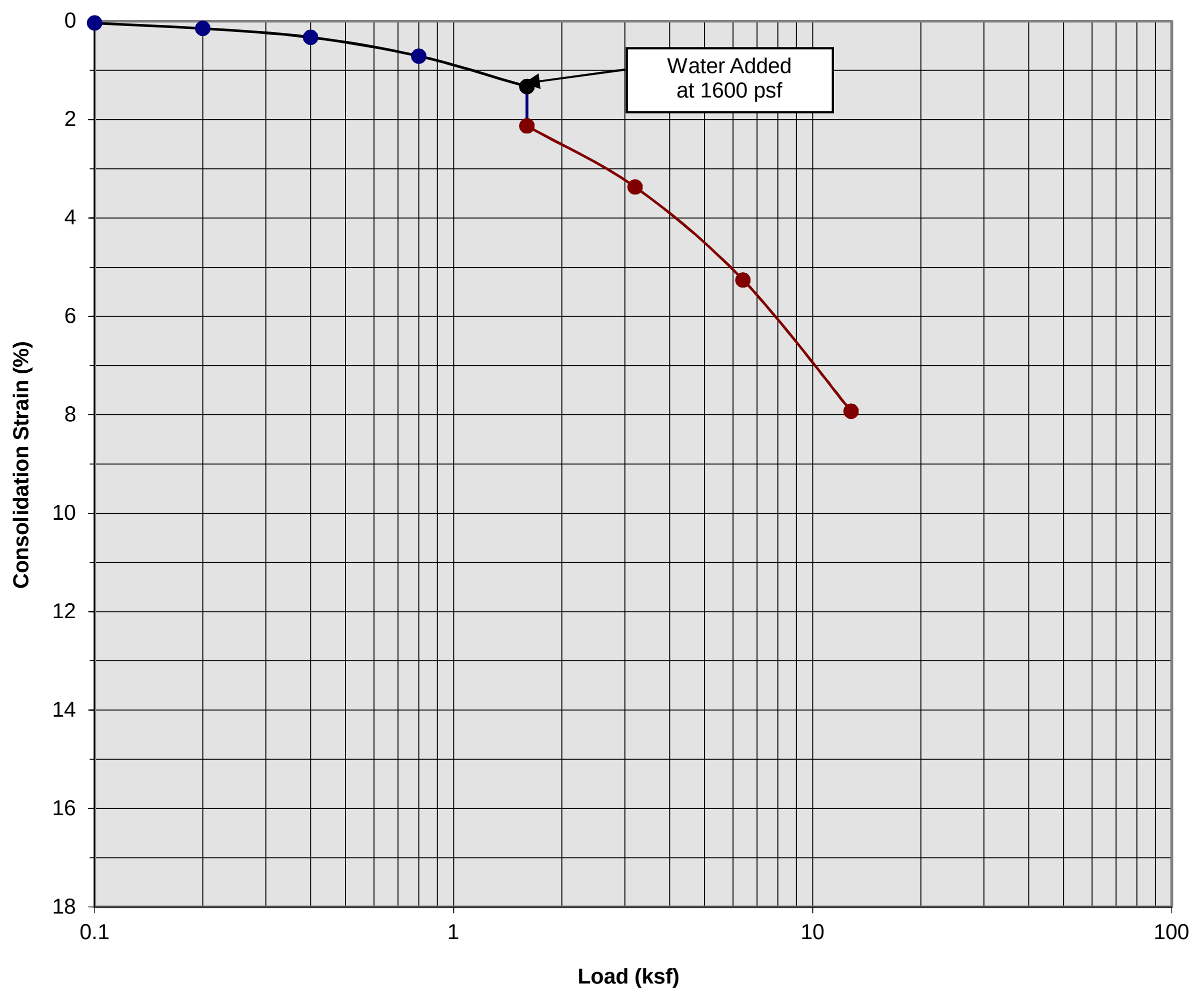
Consolidation/Collapse Test Results



Classification: Light Gray Brown Silty fine Sand

Boring Number:	B-13	Initial Moisture Content (%)	17
Sample Number:	---	Final Moisture Content (%)	27
Depth (ft)	7 to 8	Initial Dry Density (pcf)	90.6
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	101.2
Specimen Thickness (in)	1.0	Percent Collapse (%)	1.12

Consolidation/Collapse Test Results



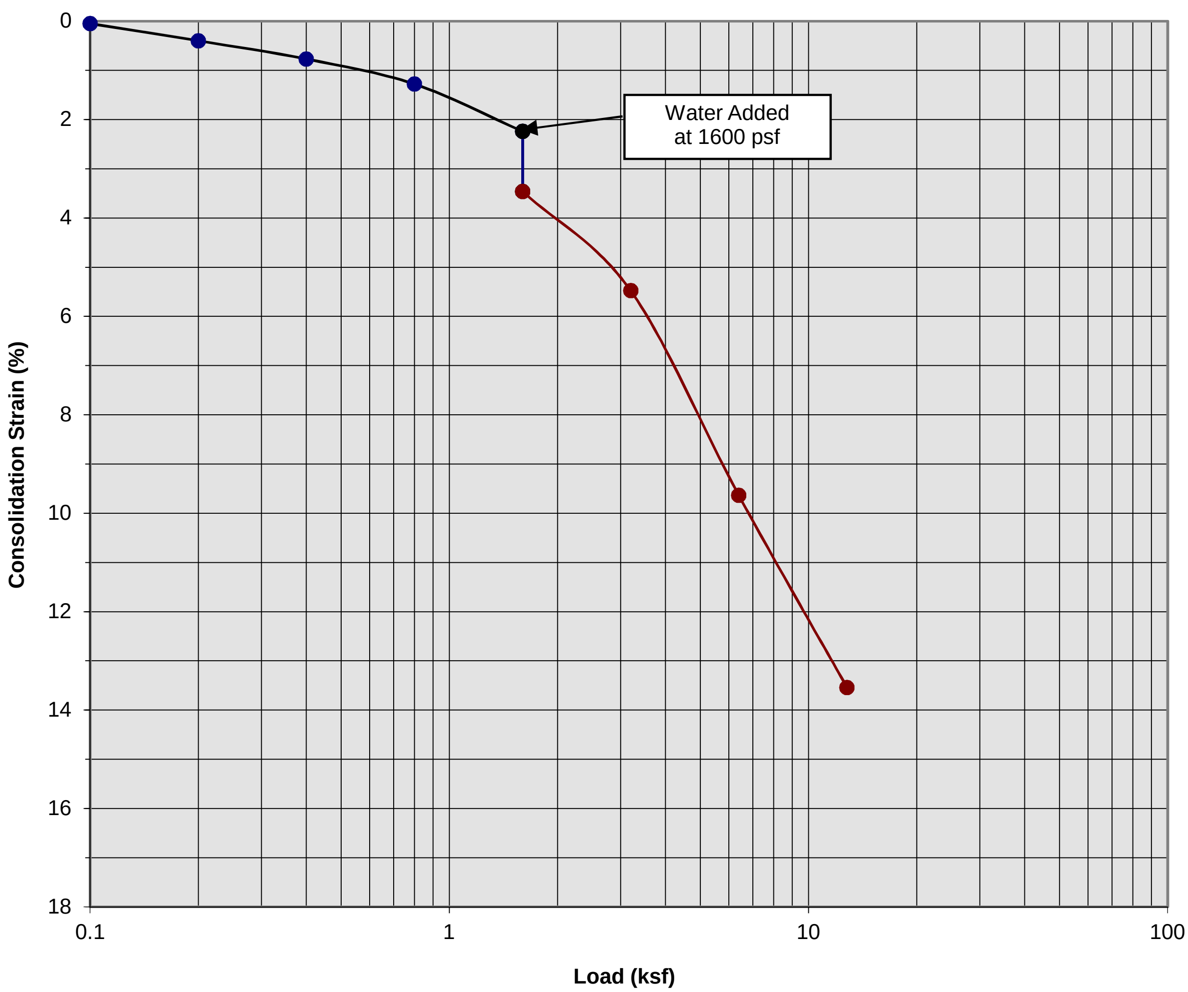
Classification: Gray Brown Silty fine Sand to fine Sandy Silt, trace Clay

Boring Number:	B-15	Initial Moisture Content (%)	14
Sample Number:	---	Final Moisture Content (%)	34
Depth (ft)	3 to 4	Initial Dry Density (pcf)	80.4
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	87.3
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.80

Proposed Commercial/Industrial Development
El Monte, California
Project No. 15G227
PLATE C- 16

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

Consolidation/Collapse Test Results

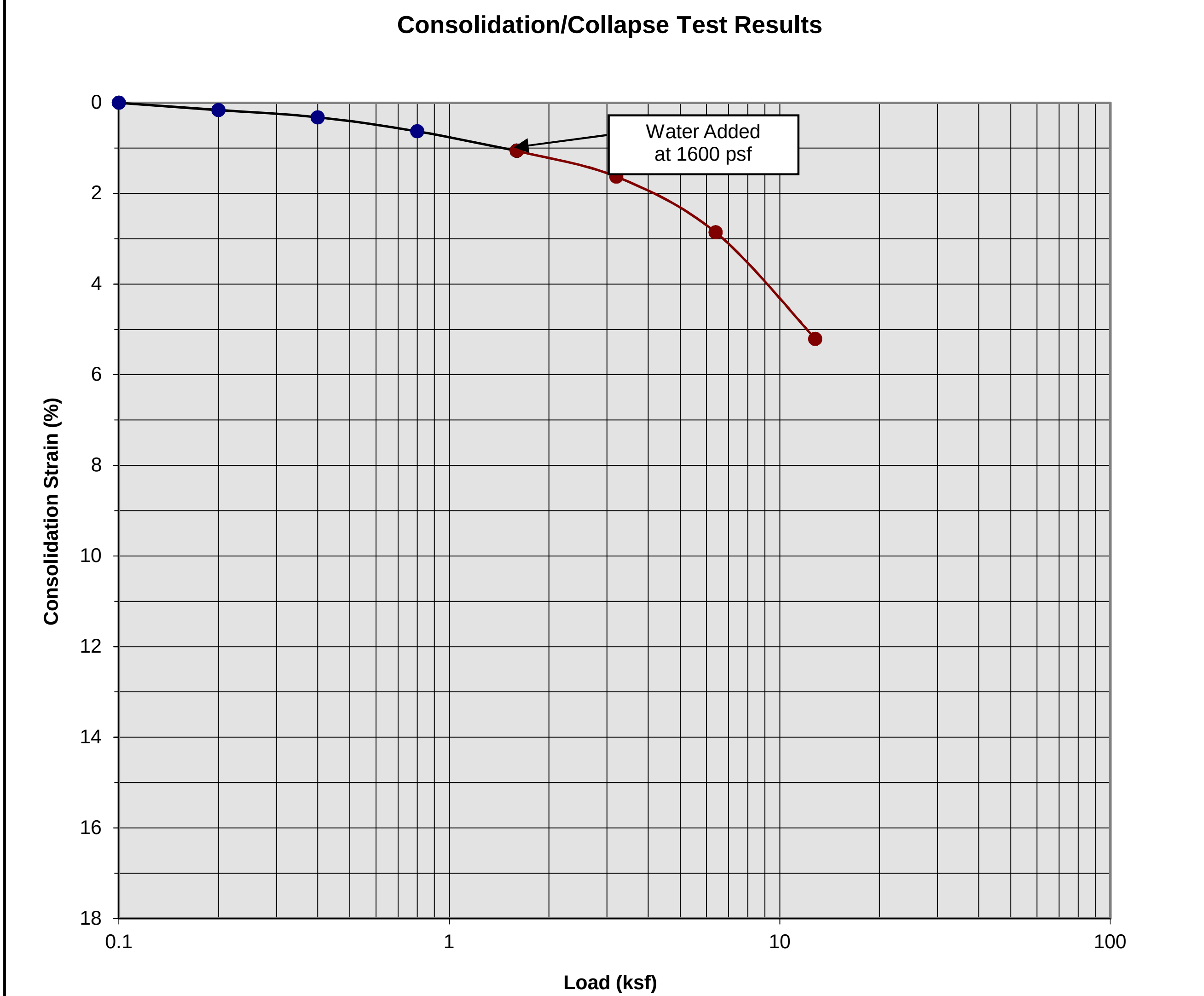


Classification: Gray Brown Silty fine Sand to fine Sandy Silt, trace Clay

Boring Number:	B-15	Initial Moisture Content (%)	29
Sample Number:	---	Final Moisture Content (%)	46
Depth (ft)	5 to 6	Initial Dry Density (pcf)	68.9
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	79.3
Specimen Thickness (in)	1.0	Percent Collapse (%)	1.22

Proposed Commercial/Industrial Development
El Monte, California
Project No. 15G227
PLATE C- 17

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

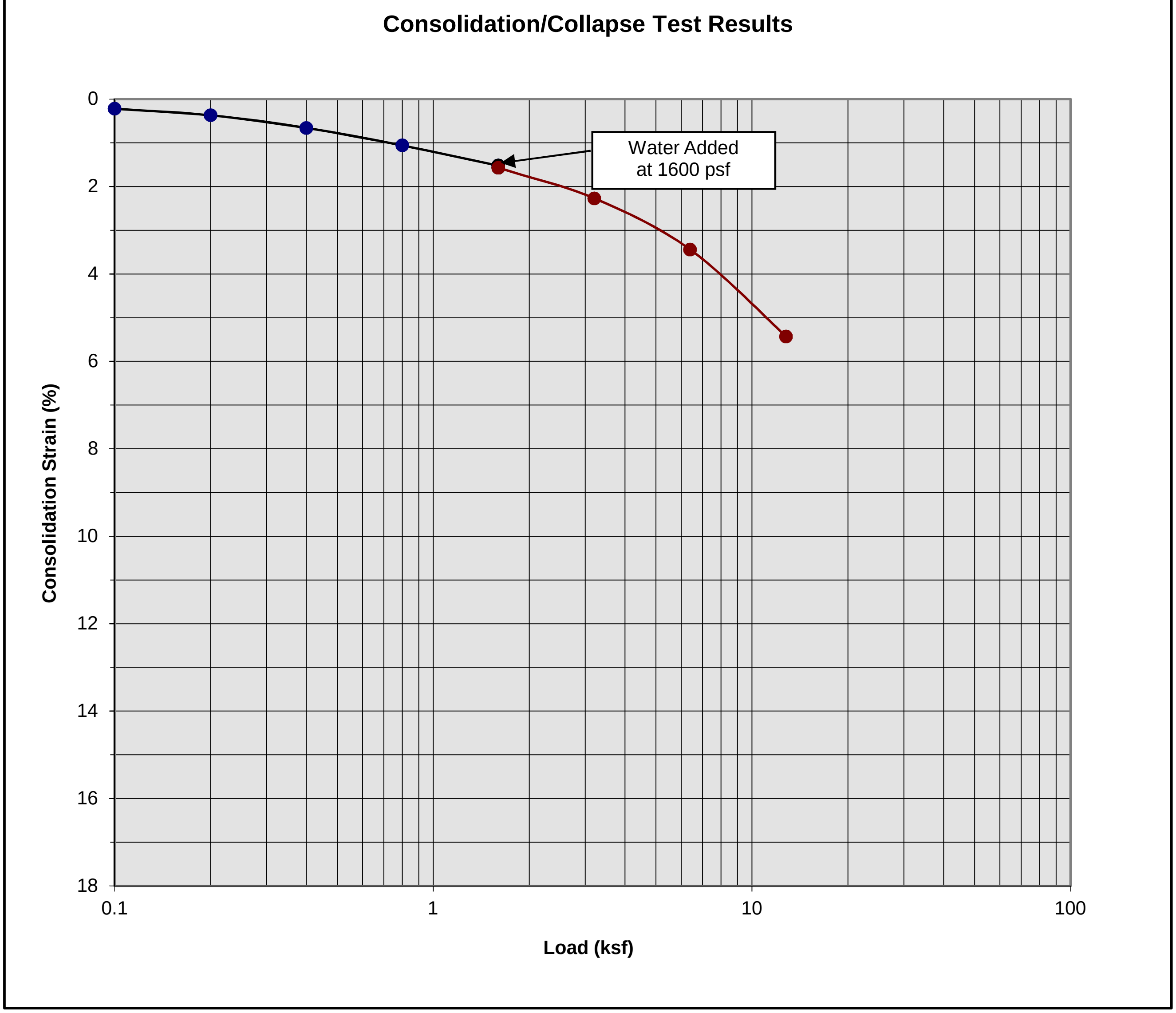


Classification: Gray Brown Silty fine Sand to fine Sandy Silt, trace Clay

Boring Number:	B-15	Initial Moisture Content (%)	22
Sample Number:	---	Final Moisture Content (%)	27
Depth (ft)	7 to 8	Initial Dry Density (pcf)	90.1
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	95.3
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.00

Proposed Commercial/Industrial Development
El Monte, California
Project No. 15G227
PLATE C- 18

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

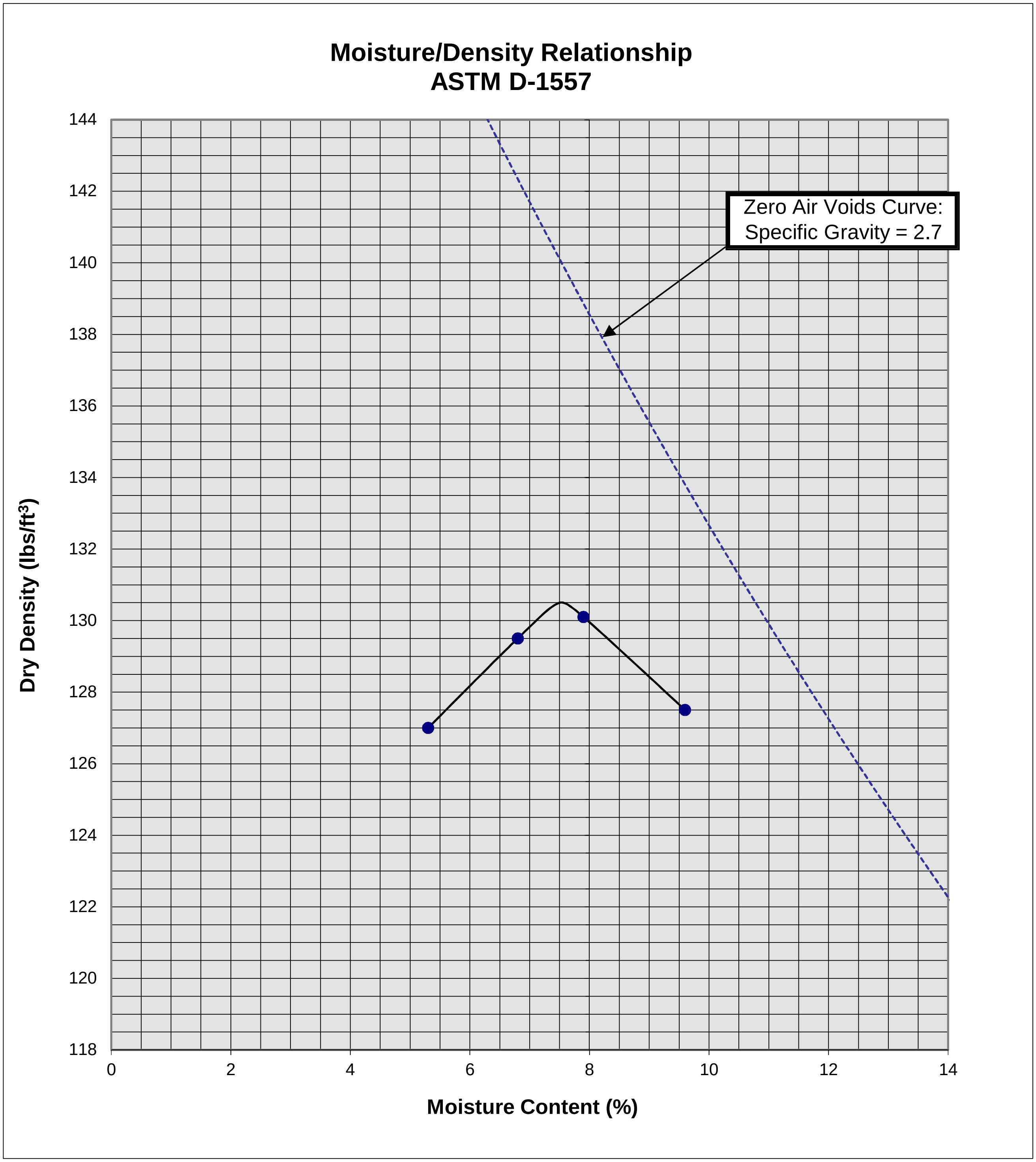


Classification: Gray Brown Silty fine Sand to fine Sandy Silt, little Clay

Boring Number:	B-15	Initial Moisture Content (%)	17
Sample Number:	---	Final Moisture Content (%)	24
Depth (ft)	9 to 10	Initial Dry Density (pcf)	97.9
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	103.9
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.05

Proposed Commercial/Industrial Development
El Monte, California
Project No. 15G227
PLATE C- 19

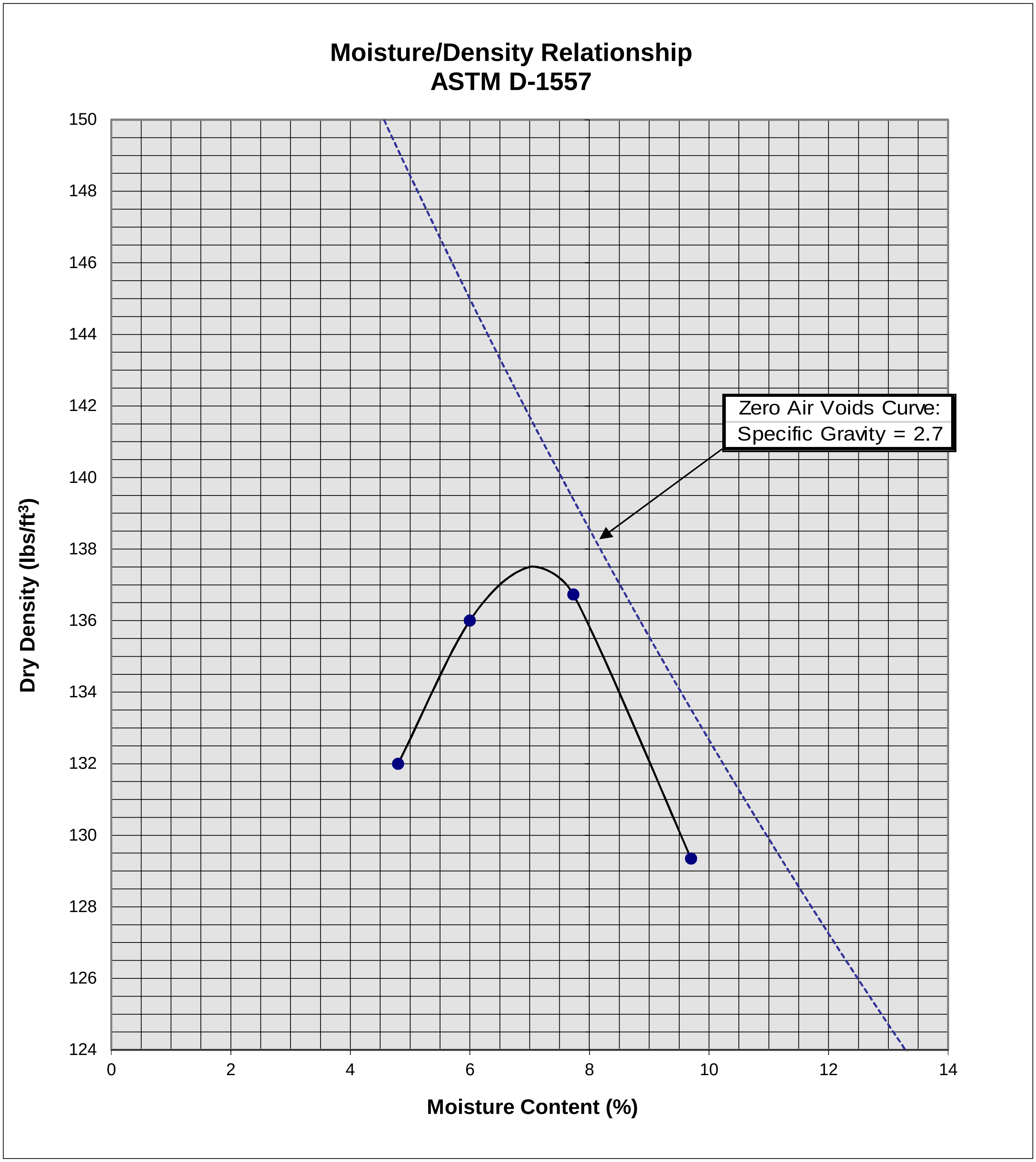
SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation



Soil ID Number	B-1 0 to 5'
Optimum Moisture (%)	7.5
Maximum Dry Density (pcf)	130.5
Soil Classification	Dark Gray Brown ine to medium Sand, trace fine to coarse Gravel little Silt

Proposed C/I Development
El Monte, California
Project No. 15G227
PLATE C-20

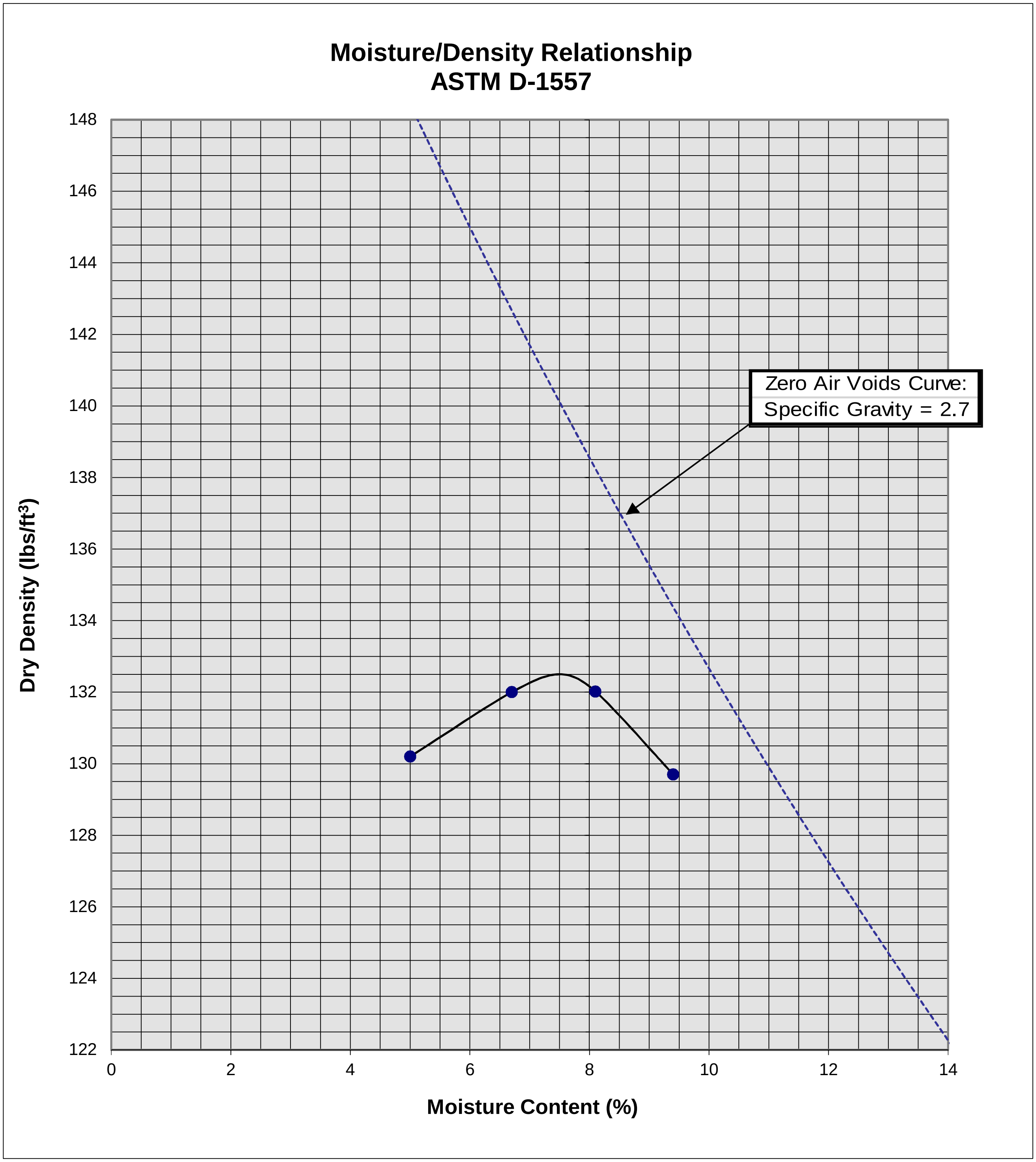
SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation



Soil ID Number	B-8 0 to 5'
Optimum Moisture (%)	7
Maximum Dry Density (pcf)	137.5
Soil Classification	Brown fine to medium Sand, trace coarse Sand little Silt, little Gravel

Proposed C/I Development
El Monte, California
Project No. 15G227
PLATE C-21

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation



Soil ID Number		B-17 0 to 5'
Optimum Moisture (%)		7.5
Maximum Dry Density (pcf)		132.5
Soil Classification	Brown Silty fine to medium Sand, trace coarse Sand, trace fine Gravel	

Proposed C/I Development
El Monte, California
Project No. 15G227
PLATE C-22

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

APPENDIX D

GRADING GUIDE SPECIFICATIONS

These grading guide specifications are intended to provide typical procedures for grading operations. They are intended to supplement the recommendations contained in the geotechnical investigation report for this project. Should the recommendations in the geotechnical investigation report conflict with the grading guide specifications, the more site specific recommendations in the geotechnical investigation report will govern.

General

- The Earthwork Contractor is responsible for the satisfactory completion of all earthwork in accordance with the plans and geotechnical reports, and in accordance with city, county, and applicable building codes.

- The Geotechnical Engineer is the representative of the Owner/Builder for the purpose of implementing the report recommendations and guidelines. These duties are not intended to relieve the Earthwork Contractor of any responsibility to perform in a workman-like manner, nor is the Geotechnical Engineer to direct the grading equipment or personnel employed by the Contractor.

- The Earthwork Contractor is required to notify the Geotechnical Engineer of the anticipated work and schedule so that testing and inspections can be provided. If necessary, work may be stopped and redone if personnel have not been scheduled in advance.

- The Earthwork Contractor is required to have suitable and sufficient equipment on the job-site to process, moisture condition, mix and compact the amount of fill being placed to the approved compaction. In addition, suitable support equipment should be available to conform with recommendations and guidelines in this report.

- Canyon cleanouts, overexcavation areas, processed ground to receive fill, key excavations, subdrains and benches should be observed by the Geotechnical Engineer prior to placement of any fill. It is the Earthwork Contractor's responsibility to notify the Geotechnical Engineer of areas that are ready for inspection.

- Excavation, filling, and subgrade preparation should be performed in a manner and sequence that will provide drainage at all times and proper control of erosion. Precipitation, springs, and seepage water encountered shall be pumped or drained to provide a suitable working surface. The Geotechnical Engineer must be informed of springs or water seepage encountered during grading or foundation construction for possible revision to the recommended construction procedures and/or installation of subdrains.

Site Preparation

- The Earthwork Contractor is responsible for all clearing, grubbing, stripping and site preparation for the project in accordance with the recommendations of the Geotechnical Engineer.

- If any materials or areas are encountered by the Earthwork Contractor which are suspected of having toxic or environmentally sensitive contamination, the Geotechnical Engineer and Owner/Builder should be notified immediately.

- Major vegetation should be stripped and disposed of off-site. This includes trees, brush, heavy grasses and any materials considered unsuitable by the Geotechnical Engineer.

- Underground structures such as basements, cesspools or septic disposal systems, mining shafts, tunnels, wells and pipelines should be removed under the inspection of the Geotechnical Engineer and recommendations provided by the Geotechnical Engineer and/or city, county or state agencies. If such structures are known or found, the Geotechnical Engineer should be notified as soon as possible so that recommendations can be formulated.

- Any topsoil, slopewash, colluvium, alluvium and rock materials which are considered unsuitable by the Geotechnical Engineer should be removed prior to fill placement.

- Remaining voids created during site clearing caused by removal of trees, foundations basements, irrigation facilities, etc., should be excavated and filled with compacted fill.

- Subsequent to clearing and removals, areas to receive fill should be scarified to a depth of 10 to 12 inches, moisture conditioned and compacted

- The moisture condition of the processed ground should be at or slightly above the optimum moisture content as determined by the Geotechnical Engineer. Depending upon field conditions, this may require air drying or watering together with mixing and/or discing.

<u>Compacted Fills</u>

- Soil materials imported to or excavated on the property may be utilized in the fill, provided each material has been determined to be suitable in the opinion of the Geotechnical Engineer. Unless otherwise approved by the Geotechnical Engineer, all fill materials shall be free of deleterious, organic, or frozen matter, shall contain no chemicals that may result in the material being classified as "contaminated," and shall be very low to non-expansive with a maximum expansion index (EI) of 50. The top 12 inches of the compacted fill should have a maximum particle size of 3 inches, and all underlying compacted fill material a maximum 6-inch particle size, except as noted below.

- All soils should be evaluated and tested by the Geotechnical Engineer. Materials with high expansion potential, low strength, poor gradation or containing organic materials may require removal from the site or selective placement and/or mixing to the satisfaction of the Geotechnical Engineer.

- Rock fragments or rocks less than 6 inches in their largest dimensions, or as otherwise determined by the Geotechnical Engineer, may be used in compacted fill, provided the distribution and placement is satisfactory in the opinion of the Geotechnical Engineer.

- Rock fragments or rocks greater than 12 inches should be taken off-site or placed in accordance with recommendations and in areas designated as suitable by the Geotechnical Engineer. These materials should be placed in accordance with Plate D-8 of these Grading Guide Specifications and in accordance with the following recommendations:

  - Rocks 12 inches or more in diameter should be placed in rows at least 15 feet apart, 15 feet from the edge of the fill, and 10 feet or more below subgrade. Spaces should be left between each rock fragment to provide for placement and compaction of soil around the fragments.

  - Fill materials consisting of soil meeting the minimum moisture content requirements and free of oversize material should be placed between and over the rows of rock or concrete. Ample water and compactive effort should be applied to the fill materials as they are placed in order that all of the voids between each of the fragments are filled and compacted to the specified density.

- Subsequent rows of rocks should be placed such that they are not directly above a row placed in the previous lift of fill. A minimum 5-foot offset between rows is recommended.

- To facilitate future trenching, oversized material should not be placed within the range of foundation excavations, future utilities or other underground construction unless specifically approved by the soil engineer and the developer/owner representative.

- Fill materials approved by the Geotechnical Engineer should be placed in areas previously prepared to receive fill and in evenly placed, near horizontal layers at about 6 to 8 inches in loose thickness, or as otherwise determined by the Geotechnical Engineer for the project.

- Each layer should be moisture conditioned to optimum moisture content, or slightly above, as directed by the Geotechnical Engineer. After proper mixing and/or drying, to evenly distribute the moisture, the layers should be compacted to at least 90 percent of the maximum dry density in compliance with ASTM D-1557-78 unless otherwise indicated.

- Density and moisture content testing should be performed by the Geotechnical Engineer at random intervals and locations as determined by the Geotechnical Engineer. These tests are intended as an aid to the Earthwork Contractor, so he can evaluate his workmanship, equipment effectiveness and site conditions. The Earthwork Contractor is responsible for compaction as required by the Geotechnical Report(s) and governmental agencies.

- Fill areas unused for a period of time may require moisture conditioning, processing and recompaction prior to the start of additional filling. The Earthwork Contractor should notify the Geotechnical Engineer of his intent so that an evaluation can be made.

- Fill placed on ground sloping at a 5-to-1 inclination (horizontal-to-vertical) or steeper should be benched into bedrock or other suitable materials, as directed by the Geotechnical Engineer. Typical details of benching are illustrated on Plates D-2, D-4, and D-5.

- Cut/fill transition lots should have the cut portion overexcavated to a depth of at least 3 feet and rebuilt with fill (see Plate D-1), as determined by the Geotechnical Engineer.

- All cut lots should be inspected by the Geotechnical Engineer for fracturing and other bedrock conditions. If necessary, the pads should be overexcavated to a depth of 3 feet and rebuilt with a uniform, more cohesive soil type to impede moisture penetration.

- Cut portions of pad areas above buttresses or stabilizations should be overexcavated to a depth of 3 feet and rebuilt with uniform, more cohesive compacted fill to impede moisture penetration.

- Non-structural fill adjacent to structural fill should typically be placed in unison to provide lateral support. Backfill along walls must be placed and compacted with care to ensure that excessive unbalanced lateral pressures do not develop. The type of fill material placed adjacent to below grade walls must be properly tested and approved by the Geotechnical Engineer with consideration of the lateral earth pressure used in the design.

Foundations

- The foundation influence zone is defined as extending one foot horizontally from the outside edge of a footing, and proceeding downward at a ½ horizontal to 1 vertical (0.5:1) inclination.

- Where overexcavation beneath a footing subgrade is necessary, it should be conducted so as to encompass the entire foundation influence zone, as described above.

- Compacted fill adjacent to exterior footings should extend at least 12 inches above foundation bearing grade. Compacted fill within the interior of structures should extend to the floor subgrade elevation.

Fill Slopes

- The placement and compaction of fill described above applies to all fill slopes. Slope compaction should be accomplished by overfilling the slope, adequately compacting the fill in even layers, including the overfilled zone and cutting the slope back to expose the compacted core

- Slope compaction may also be achieved by backrolling the slope adequately every 2 to 4 vertical feet during the filling process as well as requiring the earth moving and compaction equipment to work close to the top of the slope. Upon completion of slope construction, the slope face should be compacted with a sheepsfoot connected to a sideboom and then grid rolled. This method of slope compaction should only be used if approved by the Geotechnical Engineer.

- Sandy soils lacking in adequate cohesion may be unstable for a finished slope condition and therefore should not be placed within 15 horizontal feet of the slope face.

- All fill slopes should be keyed into bedrock or other suitable material. Fill keys should be at least 15 feet wide and inclined at 2 percent into the slope. For slopes higher than 30 feet, the fill key width should be equal to one-half the height of the slope (see Plate D-5).

- All fill keys should be cleared of loose slough material prior to geotechnical inspection and should be approved by the Geotechnical Engineer and governmental agencies prior to filling.

- The cut portion of fill over cut slopes should be made first and inspected by the Geotechnical Engineer for possible stabilization requirements. The fill portion should be adequately keyed through all surficial soils and into bedrock or suitable material. Soils should be removed from the transition zone between the cut and fill portions (see Plate D-2).

Cut Slopes

- All cut slopes should be inspected by the Geotechnical Engineer to determine the need for stabilization. The Earthwork Contractor should notify the Geotechnical Engineer when slope cutting is in progress at intervals of 10 vertical feet. Failure to notify may result in a delay in recommendations.

- Cut slopes exposing loose, cohesionless sands should be reported to the Geotechnical Engineer for possible stabilization recommendations.

- All stabilization excavations should be cleared of loose slough material prior to geotechnical inspection. Stakes should be provided by the Civil Engineer to verify the location and dimensions of the key. A typical stabilization fill detail is shown on Plate D-5.

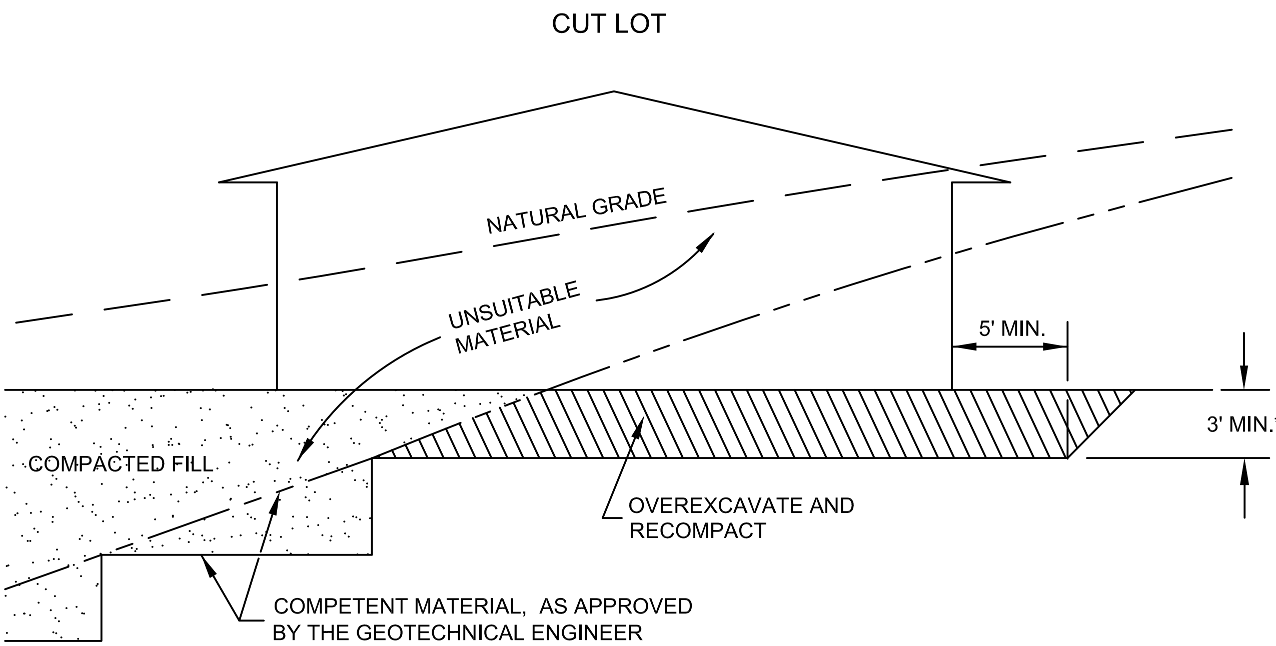
- Stabilization key excavations should be provided with subdrains. Typical subdrain details are shown on Plates D-6.

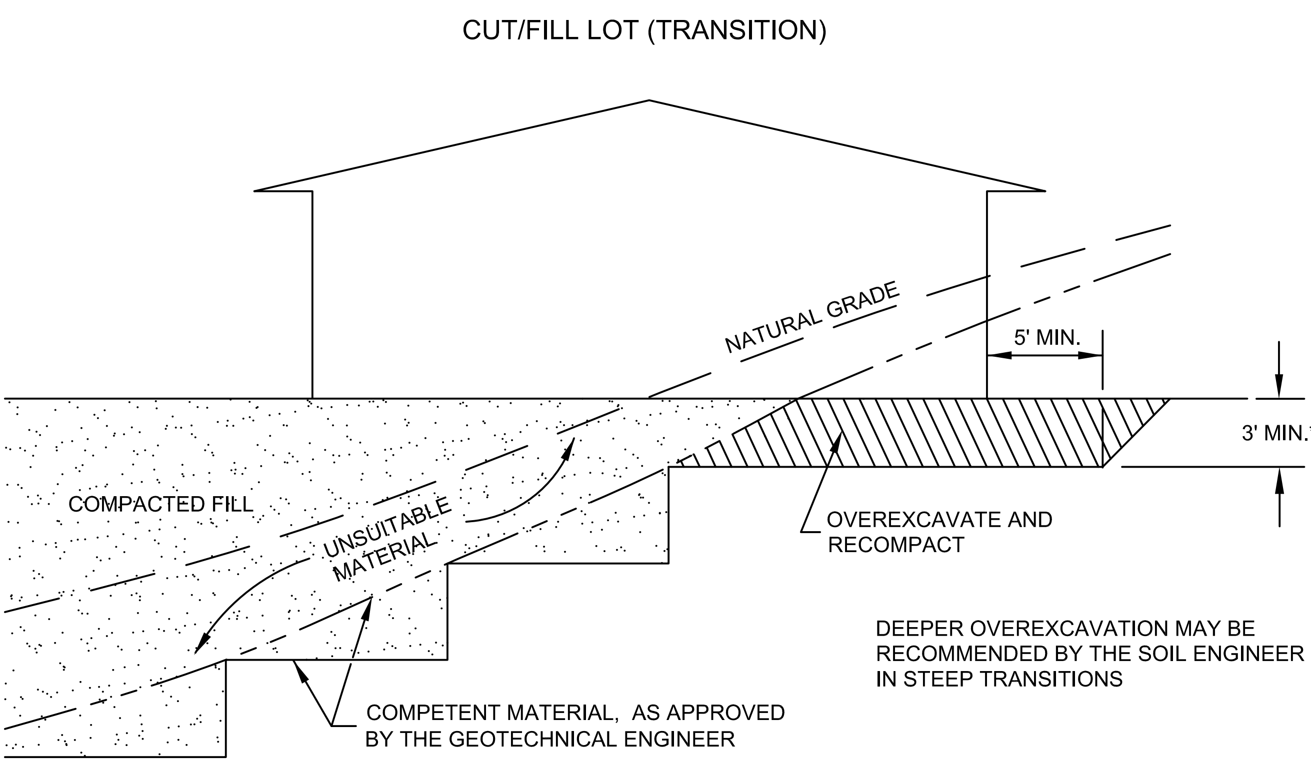
Subdrains

- Subdrains may be required in canyons and swales where fill placement is proposed. Typical subdrain details for canyons are shown on Plate D-3. Subdrains should be installed after approval of removals and before filling, as determined by the Soils Engineer.

- Plastic pipe may be used for subdrains provided it is Schedule 40 or SDR 35 or equivalent. Pipe should be protected against breakage, typically by placement in a square-cut (backhoe) trench or as recommended by the manufacturer.

- Filter material for subdrains should conform to CALTRANS Specification 68-1.025 or as approved by the Geotechnical Engineer for the specific site conditions. Clean ¾-inch crushed rock may be used provided it is wrapped in an acceptable filter cloth and approved by the Geotechnical Engineer. Pipe diameters should be 6 inches for runs up to 500 feet and 8 inches for the downstream continuations of longer runs. Four-inch diameter pipe may be used in buttress and stabilization fills.





*SEE TEXT OF REPORT FOR SPECIFIC RECOMMENDATION.
ACTUAL DEPTH OF OVEREXCAVATION MAY BE GREATER.

TRANSITION LOT DETAIL

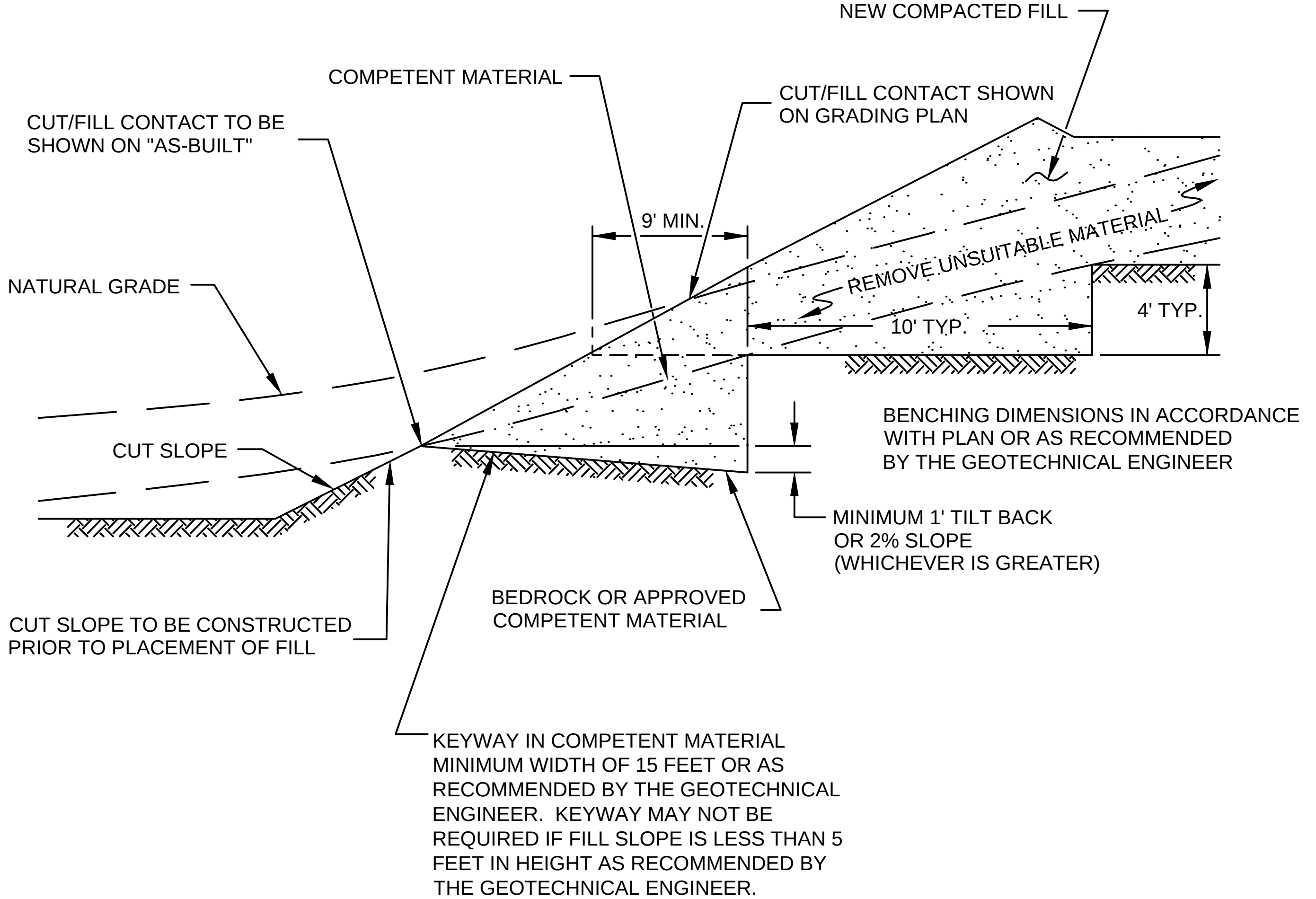
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-1

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL



FILL ABOVE CUT SLOPE DETAIL

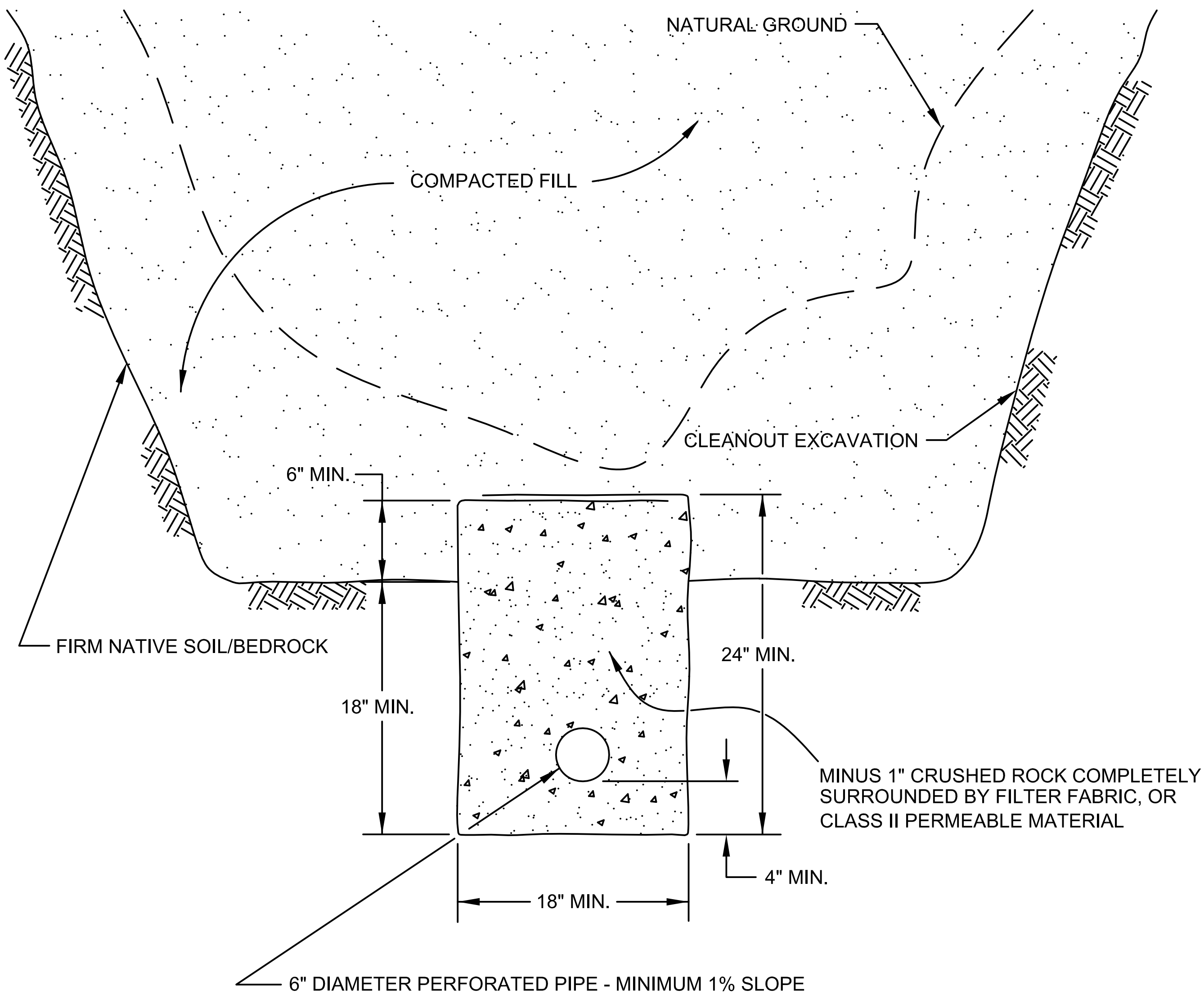
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-2

SOUTHERN CALIFORNIA GEOTECHNICAL



PIPE MATERIAL	DEPTH OF FILL OVER SUBDRAIN
ADS (CORRUGATED POLETHYLENE)	8
TRANSITE UNDERDRAIN	20
PVC OR ABS: SDR 35	35
SDR 21	100

SCHEMATIC ONLY
NOT TO SCALE

CANYON SUBDRAIN DETAIL

GRADING GUIDE SPECIFICATIONS

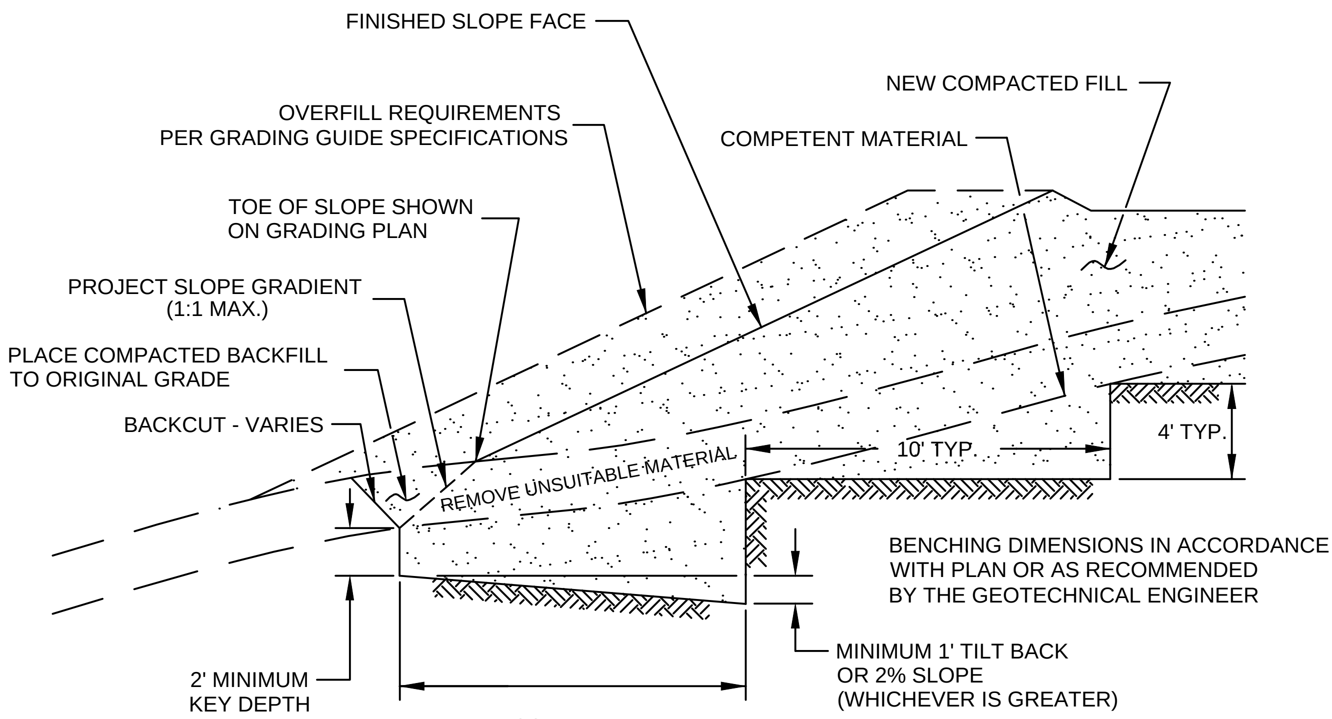
NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-3

SoCalGeo

SOUTHERN CALIFORNIA GEOTECHNICAL



KEYWAY IN COMPETENT MATERIAL. MINIMUM WIDTH OF 15 FEET OR AS RECOMMENDED BY THE GEOTECHNIAL ENGINEER. KEYWAY MAY NOT BE REQUIRED IF FILL SLOPE IS LESS THAN 5' IN HEIGHT AS RECOMMENDED BY THE GEOTECHNICAL ENGINEER.

NOTE:
BENCHING SHALL BE REQUIRED WHEN NATURAL SLOPES ARE EQUAL TO OR STEEPER THAN 5:1 OR WHEN RECOMMENDED BY THE GEOTECHNICAL ENGINEER.

FILL ABOVE NATURAL SLOPE DETAIL

GRADING GUIDE SPECIFICATIONS

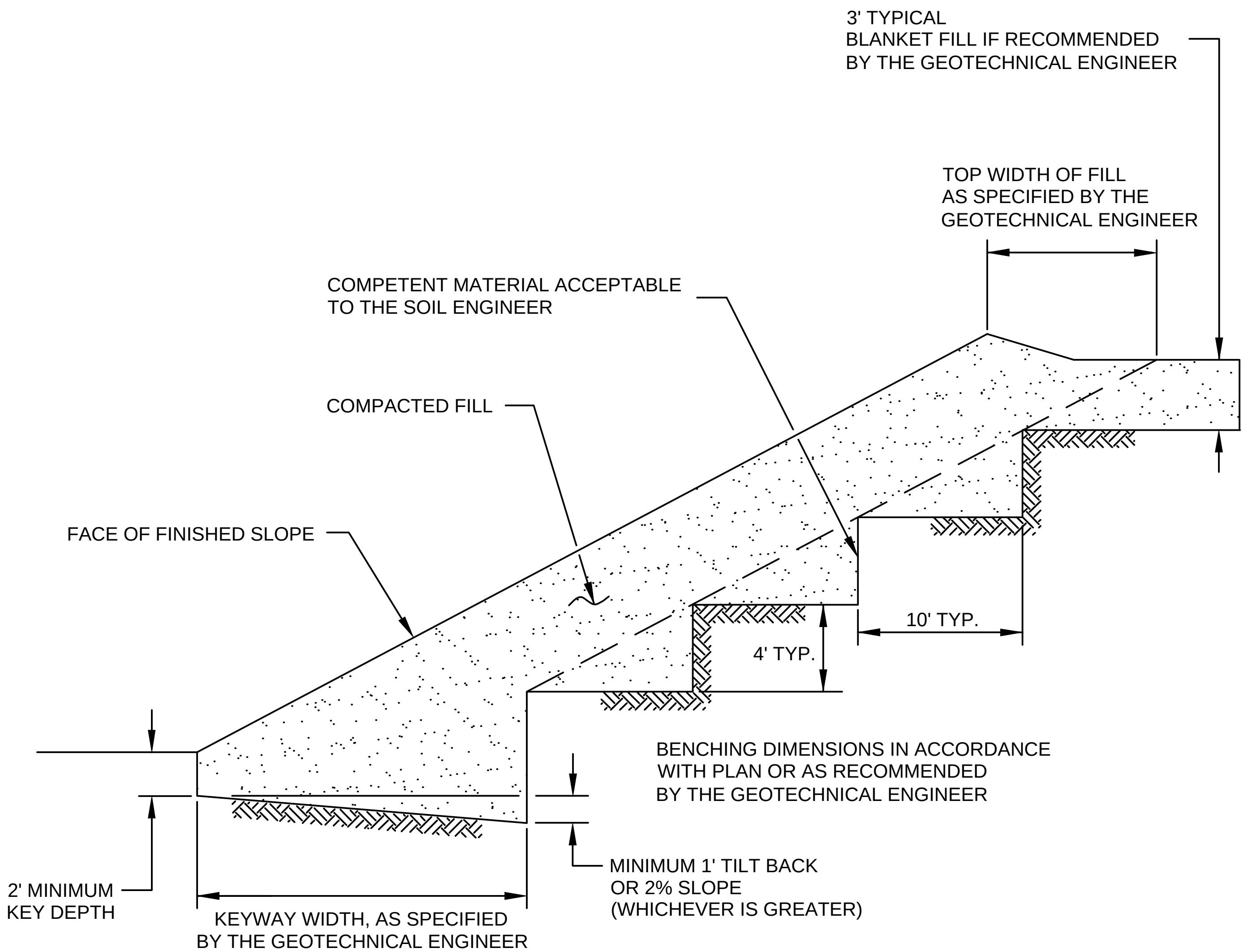
NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-4

SoCalGeo

SOUTHERN CALIFORNIA GEOTECHNICAL



STABILIZATION FILL DETAIL

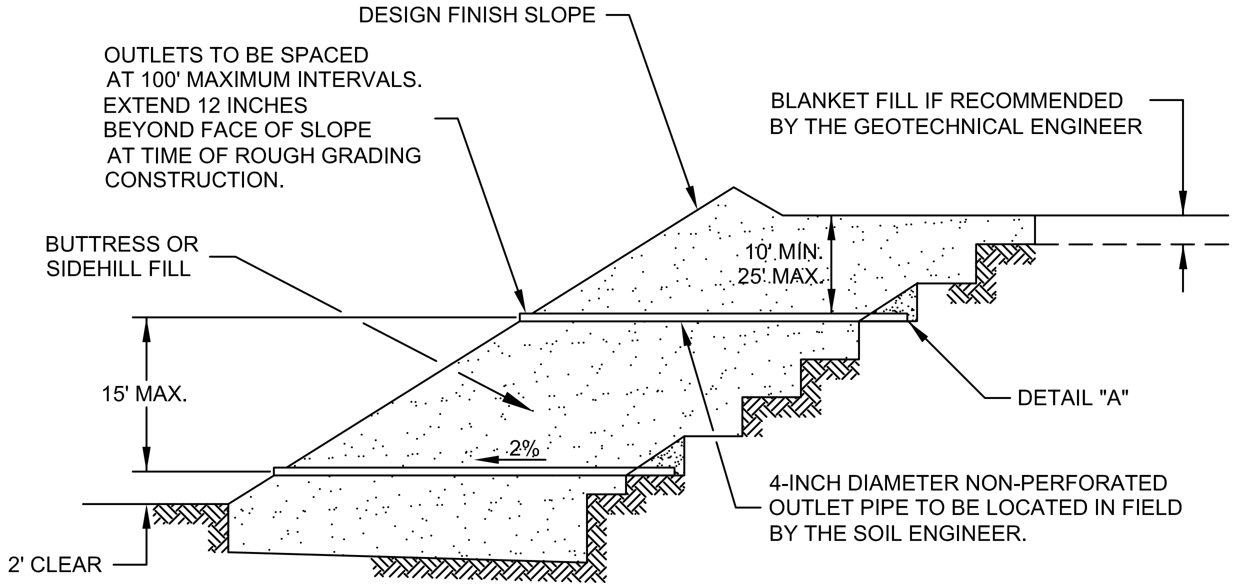
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-5

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL



"FILTER MATERIAL" TO MEET FOLLOWING SPECIFICATION OR APPROVED EQUIVALENT: (CONFORMS TO EMA STD. PLAN 323)

SIEVE SIZE	PERCENTAGE PASSING
1"	100
3/4"	90-100
3/8"	40-100
NO. 4	25-40
NO. 8	18-33
NO. 30	5-15
NO. 50	0-7
NO. 200	0-3

"GRAVEL" TO MEET FOLLOWING SPECIFICATION OR APPROVED EQUIVALENT:

SIEVE SIZE	MAXIMUM PERCENTAGE PASSING
1 1/2"	100
NO. 4	50
NO. 200	8

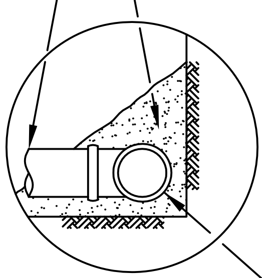
SAND EQUIVALENT = MINIMUM OF 50

OUTLET PIPE TO BE CONNECTED TO SUBDRAIN PIPE WITH TEE OR ELBOW

FILTER MATERIAL - MINIMUM OF FIVE CUBIC FEET PER FOOT OF PIPE. SEE ABOVE FOR FILTER MATERIAL SPECIFICATION.

ALTERNATIVE: IN LIEU OF FILTER MATERIAL FIVE CUBIC FEET OF GRAVEL PER FOOT OF PIPE MAY BE ENCASED IN FILTER FABRIC. SEE ABOVE FOR GRAVEL SPECIFICATION.

FILTER FABRIC SHALL BE MIRAFI 140 OR EQUIVALENT. FILTER FABRIC SHALL BE LAPPED A MINIMUM OF 12 INCHES ON ALL JOINTS.



DETAIL "A"

MINIMUM 4-INCH DIAMETER PVC SCH 40 OR ABS CLASS SDR 35 WITH A CRUSHING STRENGTH OF AT LEAST 1,000 POUNDS, WITH A MINIMUM OF 8 UNIFORMLY SPACED PERFORATIONS PER FOOT OF PIPE INSTALLED WITH PERFORATIONS ON BOTTOM OF PIPE. PROVIDE CAP AT UPSTREAM END OF PIPE. SLOPE AT 2 PERCENT TO OUTLET PIPE.

NOTES:
1. TRENCH FOR OUTLET PIPES TO BE BACKFILLED WITH ON-SITE SOIL.

SLOPE FILL SUBDRAINS

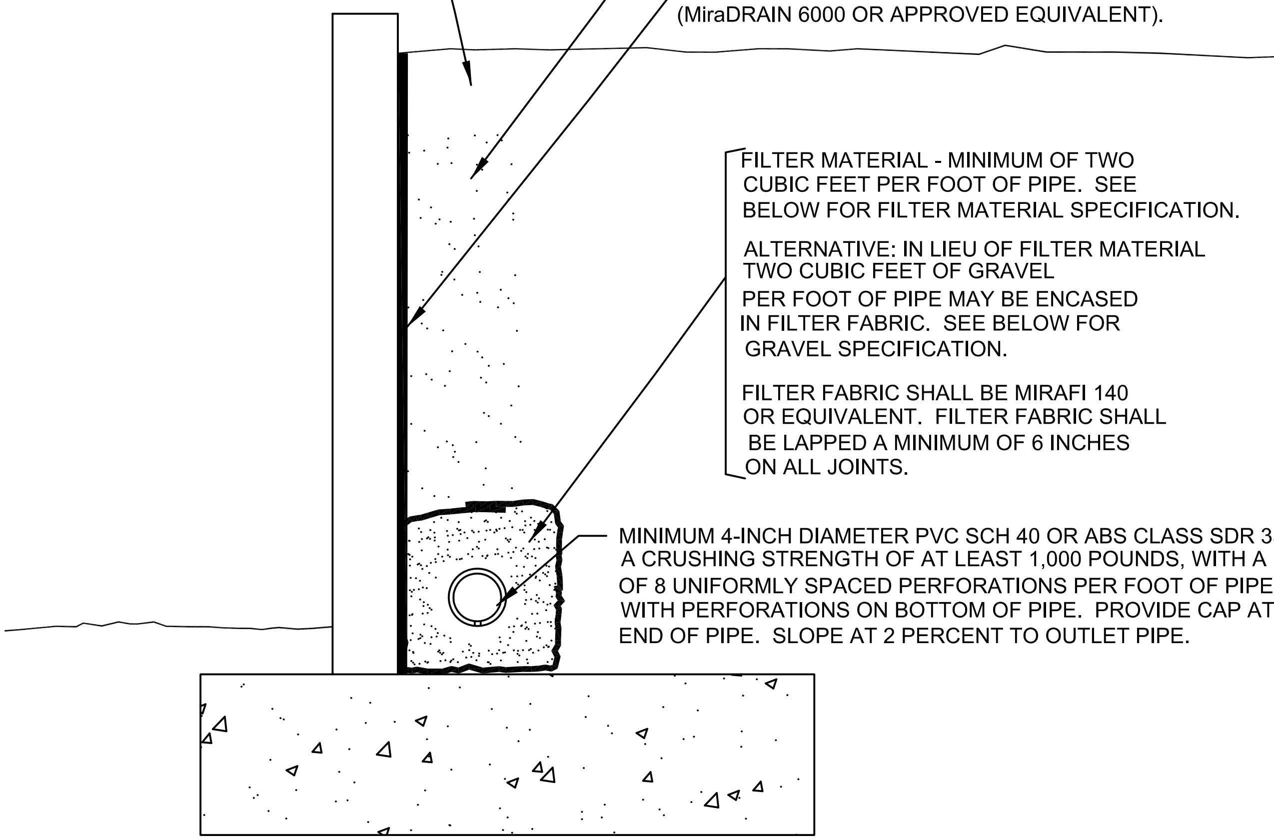
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-6

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL



"FILTER MATERIAL" TO MEET FOLLOWING SPECIFICATION OR APPROVED EQUIVALENT: (CONFORMS TO EMA STD. PLAN 323)

SIEVE SIZE	PERCENTAGE PASSING
1"	100
3/4"	90-100
3/8"	40-100
NO. 4	25-40
NO. 8	18-33
NO. 30	5-15
NO. 50	0-7
NO. 200	0-3

"GRAVEL" TO MEET FOLLOWING SPECIFICATION OR APPROVED EQUIVALENT:

SIEVE SIZE	MAXIMUM PERCENTAGE PASSING
1 1/2"	100
NO. 4	50
NO. 200	8

SAND EQUIVALENT = MINIMUM OF 50

RETAINING WALL BACKDRAINS

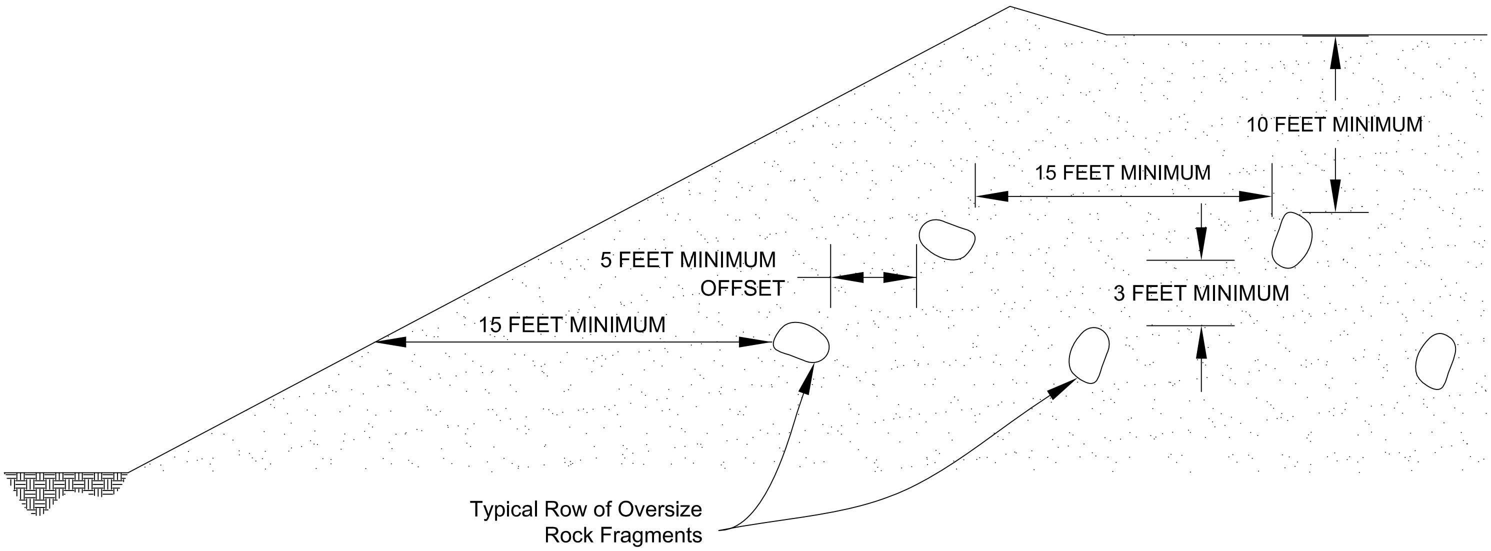
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

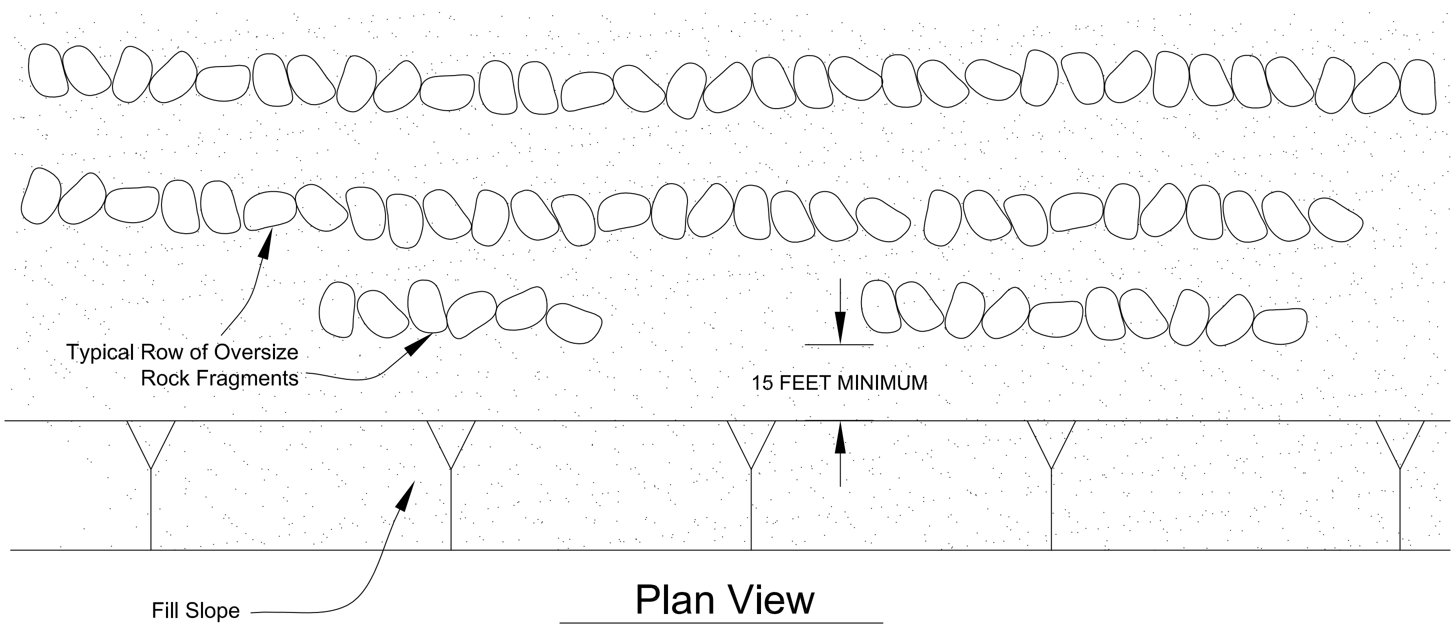
DRAWN: JAS
CHKD: GKM

PLATE D-7

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL



Section View



Plan View

PLACEMENT OF OVERSIZED MATERIAL

GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

DRAWN: PM
CHKD: GKM

PLATE D-8

SoCalGeo

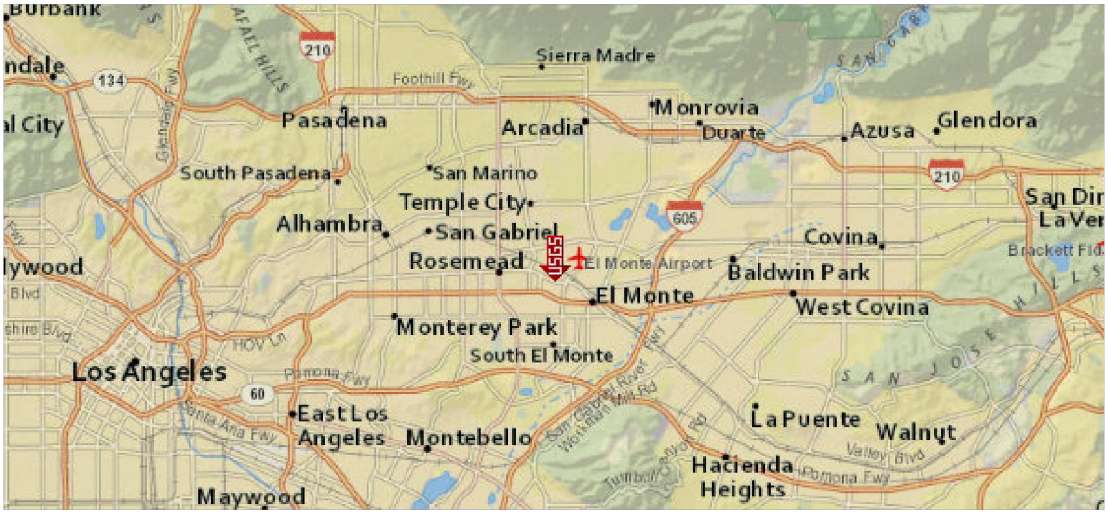
SOUTHERN CALIFORNIA GEOTECHNICAL

APPENDIX E

USGS Design Maps Summary Report

User–Specified Input

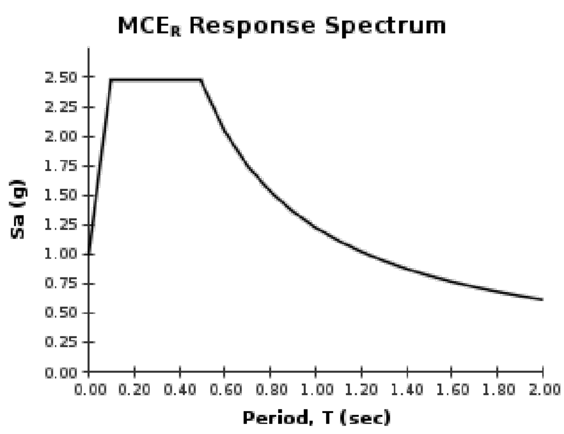
Building Code Reference Document	ASCE 7-10 Standard (which utilizes USGS hazard data available in 2008)
Site Coordinates	34.08641°N, 118.04669°W
Site Soil Classification	Site Class D – "Stiff Soil"
Risk Category	I/II/III

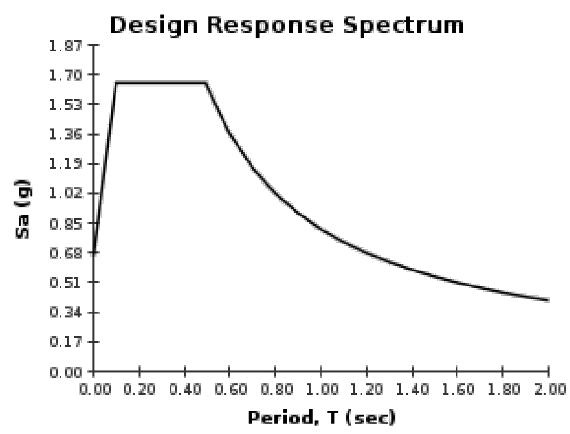


USGS–Provided Output

S_S =	2.476 g	S_{MS} =	2.476 g	S_{DS} =	1.651 g
S_1 =	0.815 g	S_{M1} =	1.223 g	S_{D1} =	0.815 g

For information on how the SS and S1 values above have been calculated from probabilistic (risk-targeted) and deterministic ground motions in the direction of maximum horizontal response, please return to the application and select the "2009 NEHRP" building code reference document.





For PGA_M, T_L, C_{RS}, and C_{R1} values, please view the detailed report.

SOURCE: U.S. GEOLOGICAL SURVEY (USGS)
<http://geohazards.usgs.gov/designmaps/us/application.php>

SEISMIC DESIGN PARAMETERS
PROPOSED COMMERCIAL INDUSTRIAL DEVELOPMENT
EL MONTE, CALIFORNIA

DRAWN: JL
CHKD: JAS

SCG PROJECT
15G227-1

PLATE E-1

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL

Section 11.8.3 — Additional Geotechnical Investigation Report Requirements for Seismic Design Categories D through F

From Figure 22-7 [4]	PGA = 0.899
Equation (11.8–1):	$PGA_M = F_{PGA}PGA = 1.000 \times 0.899 = 0.899$ g

Table 11.8–1: Site Coefficient F_{PGA}

Site Class	Mapped MCE Geometric Mean Peak Ground Acceleration, PGA				
	PGA ≤ 0.10	PGA = 0.20	PGA = 0.30	PGA = 0.40	PGA ≥ 0.50
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	$\boxed{1.0}$
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7 of ASCE 7				

Note: Use straight–line interpolation for intermediate values of PGA

For Site Class = D and PGA = 0.899 g, F_{PGA} = 1.000

Section 21.2.1.1 — Method 1 (from Chapter 21 – Site-Specific Ground Motion Procedures for Seismic Design)

From Figure 22-17 [5]	C_{RS} = 0.964
From Figure 22-18 [6]	C_{R1} = 0.982

SOURCE: U.S. GEOLOGICAL SURVEY (USGS)
<http://geohazards.usgs.gov/designmaps/us/application.php>

SEISMIC DESIGN PARAMETERS
PROPOSED COMMERCIAL INDUSTRIAL DEVELOPMENT
EL MONTE, CALIFORNIA

DRAWN: JL
CHKD: JAS

SCG PROJECT
15G227-1

PLATE E-2

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL

APPENDIX F

LIQUEFACTION EVALUATION

Project Name	Proposed C/I Development	MCE_G Design Acceleration	0.899 (g)
Project Location	El Monte, CA	Design Magnitude	6.66
Project Number	15G227	Historic High Depth to Groundwater	10 (ft)
Engineer	DWN	Depth to Groundwater at Time of Drilling	60 (ft)
		Borehole Diameter	6 (in)
Boring No.	B-1		

Sample Depth (ft)	Depth to Top of Layer (ft)	Depth to Bottom of Layer (ft)	Depth to Midpoint (ft)	Uncorrected SPT N-Value	Unit Weight of Soil (pcf)	Fines Content (%)	Energy Correction	C_B	C_S	C_N	Rod Length Correction	$(N_1)_{60}$	$(N_1)_{60CS}$	Overburden Stress (σ_o) (psf)	Eff. Overburden Stress (Hist. Water) (σ_o') (psf)	Eff. Overburden Stress (Curr. Water) (σ_o') (psf)	Stress Reduction Coefficient (r_d)	MSF	Ks	Cyclic Resistance Ratio (M=7.5)	Cyclic Resistance Ratio (M=6.66)	Cyclic Stress Ratio Induced by Design Earthquake	Factor of Safety	Comments
							(1)	(2)	(3)	(4)	(5)	(6)	(7)				(8)	(9)	(10)	(11)	(12)	(13)		
7	0	10	5		120		1.3	1.05	1.1	1.70	0.75	0.0	0.0	600	600	600	0.99	1.03	1.07	0.06	0.07	N/A	N/A	Above Water Table
9.5	10	12	11	11	120	3	1.3	1.05	1.16	1.25	0.75	16.4	16.4	1320	1258	1320	0.96	1.11	1.06	0.17	0.20	0.59	0.34	**Liquefiable**
14.5	12	17	14.5	18	120	3	1.3	1.05	1.29	1.08	0.85	29.0	29.0	1740	1459	1740	0.95	1.29	1.07	0.43	0.59	0.66	0.89	**Liquefiable**
19.5	17	22	19.5	10	120	36	1.3	1.05	1.14	0.96	0.95	14.2	19.7	2340	1747	2340	0.92	1.15	1.02	0.20	0.24	0.72	0.33	**Liquefiable**
24.5	22	27	24.5	15	120	14	1.3	1.05	1.2	0.87	0.95	20.5	23.4	2940	2035	2940	0.89	1.20	1	0.26	0.31	0.75	0.41	**Liquefiable**
29.5	27	32	29.5	35	120		1.3	1.05	1.3	0.89	0.95	52.5	52.5	3540	2323	3540	0.86	1.37	0.97	2.00	2.00	0.77	2.61	Non-Liquefiable
34.5	32	37	34.5	46	120		1.3	1.05	1.3	0.92	1	75.5	75.5	4140	2611	4140	0.83	1.37	0.94	2.00	2.00	0.77	2.60	Non-Liquefiable
38.5	37	42	39.5	48	120		1.3	1.05	1.3	0.92	1	78.3	78.3	4740	2899	4740	0.80	1.37	0.9	2.00	2.00	0.76	2.62	Non-Liquefiable
44.5	42	47	44.5	46	120		1.3	1.05	1.3	0.89	1	72.3	72.3	5340	3187	5340	0.77	1.37	0.88	2.00	2.00	0.75	2.66	Non-Liquefiable
49.5	47	50	48.5	29	120	26	1.3	1.05	1.3	0.76	1	39.0	44.2	5820	3418	5820	0.74	1.37	0.86	2.00	2.00	0.74	2.70	Non-Liquefiable

Notes:

(1) Energy Correction for N_{90} of automatic hammer to standard N_{60}

(2) Borehole Diameter Correction (Skempton, 1986)

(3) Correction for split-spoon sampler with room for liners, but liners are absent, (Seed et al., 1984, 2001)

(4) Overburden Correction, Caluclated by Eq. 39 (Boulanger and Idriss, 2008)

(5) Rod Length Correction for Samples <10 m in depth

(6) N-value corrected for energy, borehole diameter, sampler with absent liners, rod length, and overburden

(7) N-value corrected for fines content per Eqs. 75 and 76 (Boulanger and Idriss, 2008)

(8) Stress Reduction Coefficient calculated by Eq. 22 (Boulanger and Idriss, 2008)

(9) Magnitude Scaling Factor calculated by Eqns. A.8 & A.10 (Boulanger and Idriss, 2014)

(10) Overburden Correction Factor calcuated by Eq. 54 (Boulanger and Idriss, 2008)

(11) Calcuated by Eq. 70 (Boulanger and Idriss, 2008)

(12) Calcuated by Eq. 72 (Boulanger and Idriss, 2008)

(13) Calcuated by Eq. 25 (Boulanger and Idriss, 2008)

LIQUEFACTION INDUCED SETTLEMENTS

Project Name	Proposed C/I Development
Project Location	El Monte, CA
Project Number	15G227
Engineer	DWN

Boring No. B-1

Sample Depth (ft)	Depth to Top of Layer (ft)	Depth to Bottom of Layer (ft)	Depth to Midpoint (ft)	$(N_1)_{60}$	DN for fines content	$(N_1)_{60\text{-}CS}$	Liquefaction Factor of Safety	Limiting Shear Strain γ_{min}	Parameter Fα	Maximum Shear Strain γ_{max}	Height of Layer		Vertical Reconsolidation Strain ε_v		Total Deformation of Layer (in)	Comments
				(1)	(2)	(3)	(4)	(5)	(6)	(7)			(8)			
7	0	10	5	0.0	0.0	0.0	N/A	0.50	0.95	0.00	10.00		0.000		0.00	Above Water Table
9.5	10	12	11	16.4	0.0	16.4	0.34	0.24	0.69	0.24	2.00		0.027		0.65	Liquefiable
14.5	12	17	14.5	29.0	0.0	29.0	0.89	0.05	-0.02	0.04	5.00		0.009		0.54	Liquefiable
19.5	17	22	19.5	14.2	5.5	19.7	0.33	0.16	0.53	0.16	5.00		0.000		0.00	Liquefiable
24.5	22	27	24.5	20.5	2.9	23.4	0.41	0.11	0.33	0.11	5.00		0.020		1.21	Liquefiable
29.5	27	32	29.5	52.5	0.0	52.5	2.61	0.00	-1.79	0.00	5.00		0.000		0.00	Non-Liquefiable
34.5	32	37	34.5	75.5	0.0	75.5	2.60	0.00	-3.79	0.00	5.00		0.000		0.00	Non-Liquefiable
38.5	37	42	39.5	78.3	0.0	78.3	2.62	0.00	-4.04	0.00	5.00		0.000		0.00	Non-Liquefiable
44.5	42	47	44.5	72.3	0.0	72.3	2.66	0.00	-3.50	0.00	5.00		0.000		0.00	Non-Liquefiable
49.5	47	50	48.5	39.0	5.1	44.2	2.70	0.00	-1.12	0.00	3.00		0.000		0.00	Non-Liquefiable
											Total Deformation (in)				2.39	

Notes:

(1) $(N_1)_{60}$ calculated previously for the individual layer

(2) Correction for fines content per Equation 76 (Boulanger and Idriss, 2008)

(3) Corrected $(N_1)_{60}$ for fines content

(4) Factor of Safety against Liquefaction, calculated previously for the individual layer

(5) Calcuated by Eq. 86 (Boulanger and Idriss, 2008)

(6) Calcuated by Eq. 89 (Boulanger and Idriss, 2008)

(7) Calcuated by Eqs. 90, 91, and 92 (Boulanger and Idriss, 2008)

(8) Voumetric Strain Induced in a Liquefiable Layer, Calcuated by Eq. 96 (Boulanger and Idriss, 2008)
(Strain N/A if Factor of Safety against Liquefaction > 1.3)

LIQUEFACTION EVALUATION

Project Name	Proposed C/I Development	MCE$_G$ Design Acceleration	0.899	(g)
Project Location	El Monte, CA	Design Magnitude	6.66	
Project Number	15G227	Historic High Depth to Groundwater	20	(ft)
Engineer	DWN	Depth to Groundwater at Time of Drilling	60	(ft)
		Borehole Diameter	6	(in)
Boring No.	B-8			

Sample Depth (ft)	Depth to Top of Layer (ft)	Depth to Bottom of Layer (ft)	Depth to Midpoint (ft)	Uncorrected SPT N-Value	Unit Weight of Soil (pcf)	Fines Content (%)	Energy Correction	C_B	C_S	C_N	Rod Length Correction	$(N_1)_{60}$	$(N_1)_{60CS}$	Overburden Stress (σ_o) (psf)	Eff. Overburden Stress (Hist. Water) (σ_o') (psf)	Eff. Overburden Stress (Curr. Water) (σ_o') (psf)	Stress Reduction Coefficient (r_d)	MSF	Ks	Cyclic Resistance Ratio (M=7.5)	Cyclic Resistance Ratio (M=6.66)	Cyclic Stress Ratio Induced by Design Earthquake	Factor of Safety	Comments
							(1)	(2)	(3)	(4)	(5)	(6)	(7)				(8)	(9)	(10)	(11)	(12)	(13)		
7	0	10	5	10	120		1.3	1.05	1.21	1.70	0.75	21.1	21.1	600	600	600	0.99	1.17	1.1	0.22	0.28	N/A	N/A	Above Water Table
9.5	10	12	11	10	120	50	1.3	1.05	1.14	1.23	0.75	14.4	20.0	1320	1320	1320	0.96	1.15	1.06	0.21	0.25	0.56	N/A	Above Water Table
14.5	12	17	14.5	12	120	4	1.3	1.05	1.18	1.09	0.85	18.0	18.0	1740	1740	1740	0.95	1.13	1.02	0.18	0.21	0.55	N/A	Above Water Table
19.5	17	20	18.5	18	120	54	1.3	1.05	1.3	0.98	0.95	29.8	35.4	2220	2220	2220	0.93	1.37	0.99	1.22	1.65	0.54	N/A	Above Water Table
19.5	20	22	21	18	120	6	1.3	1.05	1.28	0.94	0.95	28.0	28.0	2520	2458	2520	0.91	1.27	0.97	0.38	0.47	0.55	0.87	**Liquefiable**
24.5	22	27	24.5	12	120	6	1.3	1.05	1.15	0.85	0.95	15.3	15.3	2940	2659	2940	0.89	1.10	0.97	0.16	0.17	0.58	0.30	**Liquefiable**
29.5	27	32	29.5	14	120	36	1.3	1.05	1.17	0.81	0.95	17.1	22.7	3540	2947	3540	0.86	1.19	0.95	0.24	0.28	0.60	0.46	**Liquefiable**
34.5	32	34.5	33.3	27	120		1.3	1.05	1.3	0.83	1	39.6	39.6	3990	3163	3990	0.84	1.37	0.88	2.00	2.00	0.62	3.24	Non-Liquefiable
34.5	34.5	37	35.8	27	120	19	1.3	1.05	1.3	0.82	1	39.4	43.7	4290	3307	4290	0.82	1.37	0.87	2.00	2.00	0.62	3.21	Non-Liquefiable
39.5	37	42	39.5	27	120	23	1.3	1.05	1.3	0.80	1	38.2	43.1	4740	3523	4740	0.80	1.37	0.85	2.00	2.00	0.63	3.18	Non-Liquefiable
44.5	42	47	44.5	42	120		1.3	1.05	1.3	0.85	1	63.6	63.6	5340	3811	5340	0.77	1.37	0.82	2.00	2.00	0.63	3.18	Non-Liquefiable
49.5	47	50	48.5	39	120		1.3	1.05	1.3	0.81	1	56.0	56.0	5820	4042	5820	0.74	1.37	0.81	2.00	2.00	0.63	3.20	Non-Liquefiable

Notes:

(1) Energy Correction for N_{90} of automatic hammer to standard N_{60}

(2) Borehole Diameter Correction (Skempton, 1986)

(3) Correction for split-spoon sampler with room for liners, but liners are absent, (Seed et al., 1984, 2001)

(4) Overburden Correction, Caluclated by Eq. 39 (Boulanger and Idriss, 2008)

(5) Rod Length Correction for Samples <10 m in depth

(6) N-value corrected for energy, borehole diameter, sampler with absent liners, rod length, and overburden

(7) N-value corrected for fines content per Eqs. 75 and 76 (Boulanger and Idriss, 2008)

(8) Stress Reduction Coefficient calculated by Eq. 22 (Boulanger and Idriss, 2008)

(9) Magnitude Scaling Factor calculated by Eqns. A.8 & A.10 (Boulanger and Idriss, 2014)

(10) Overburden Correction Factor calcuated by Eq. 54 (Boulanger and Idriss, 2008)

(11) Calcuated by Eq. 70 (Boulanger and Idriss, 2008)

(12) Calcuated by Eq. 72 (Boulanger and Idriss, 2008)

(13) Calcuated by Eq. 25 (Boulanger and Idriss, 2008)

LIQUEFACTION INDUCED SETTLEMENTS

Project Name	Proposed C/I Development
Project Location	El Monte, CA
Project Number	15G227
Engineer	DWN

Boring No. B-8

Sample Depth (ft)	Depth to Top of Layer (ft)	Depth to Bottom of Layer (ft)	Depth to Midpoint (ft)	$(N_1)_{60}$	DN for fines content	$(N_1)_{60\text{-}CS}$	Liquefaction Factor of Safety	Limiting Shear Strain γ_{min}	Parameter Fα	Maximum Shear Strain γ_{max}	Height of Layer		Vertical Reconsolidation Strain ε_v		Total Deformation of Layer (in)	Comments
				(1)	(2)	(3)	(4)	(5)	(6)	(7)			(8)			
7	0	10	5	21.1	0.0	21.1	N/A	0.14	0.46	0.00	10.00		0.000		0.00	Above Water Table
9.5	10	12	11	14.4	5.6	20.0	N/A	0.16	0.52	0.00	2.00		0.000		0.00	**Above Water Table**
14.5	12	17	14.5	18.0	0.0	18.0	N/A	0.20	0.62	0.00	5.00		0.000		0.00	**Above Water Table**
19.5	17	20	18.5	29.8	5.6	35.4	N/A	0.02	-0.47	0.00	3.00		0.000		0.00	Above Water Table
19.5	20	22	21	28.0	0.0	28.0	0.87	0.06	0.04	0.05	2.00		0.010		0.24	**Liquefiable**
24.5	22	27	24.5	15.3	0.0	15.3	0.30	0.27	0.74	0.27	5.00		0.028		1.70	**Liquefiable**
29.5	27	32	29.5	17.1	5.5	22.7	0.46	0.12	0.37	0.12	5.00		0.021		1.24	**Liquefiable**
34.5	32	34.5	33.3	39.6	0.0	39.6	3.24	0.01	-0.77	0.00	2.50		0.000		0.00	Non-Liquefiable
34.5	34.5	37	35.8	39.4	4.3	43.7	3.21	0.00	-1.09	0.00	2.50		0.000		0.00	Non-Liquefiable
39.5	37	42	39.5	38.2	4.9	43.1	3.18	0.00	-1.04	0.00	5.00		0.000		0.00	Non-Liquefiable
44.5	42	47	44.5	63.6	0.0	63.6	3.18	0.00	-2.73	0.00	5.00		0.000		0.00	Non-Liquefiable
49.5	47	50	48.5	56.0	0.0	56.0	3.20	0.00	-2.08	0.00	3.00		0.000		0.00	Non-Liquefiable
											Total Deformation (in)				3.18	

Notes:

(1) $(N_1)_{60}$ calculated previously for the individual layer

(2) Correction for fines content per Equation 76 (Boulanger and Idriss, 2008)

(3) Corrected $(N_1)_{60}$ for fines content

(4) Factor of Safety against Liquefaction, calculated previously for the individual layer

(5) Calcuated by Eq. 86 (Boulanger and Idriss, 2008)

(6) Calcuated by Eq. 89 (Boulanger and Idriss, 2008)

(7) Calcuated by Eqs. 90, 91, and 92 (Boulanger and Idriss, 2008)

(8) Voumetric Strain Induced in a Liquefiable Layer, Calcuated by Eq. 96 (Boulanger and Idriss, 2008)
(Strain N/A if Factor of Safety against Liquefaction > 1.3)

LIQUEFACTION EVALUATION

Project Name	Proposed C/I Development	MCE$_G$ Design Acceleration	0.899	(g)
Project Location	El Monte, CA	Design Magnitude	6.66	
Project Number	15G227	Historic High Depth to Groundwater	10	(ft)
Engineer	DWN	Depth to Groundwater at Time of Drilling	60	(ft)
		Borehole Diameter	6	(in)
Boring No.	B-9			

Sample Depth (ft)	Depth to Top of Layer (ft)	Depth to Bottom of Layer (ft)	Depth to Midpoint (ft)	Uncorrected SPT N-Value	Unit Weight of Soil (pcf)	Fines Content (%)	Energy Correction	C_B	C_S	C_N	Rod Length Correction	$(N_1)_{60}$	$(N_1)_{60CS}$	Overburden Stress (σ_o) (psf)	Eff. Overburden Stress (Hist. Water) (σ_o') (psf)	Eff. Overburden Stress (Curr. Water) (σ_o') (psf)	Stress Reduction Coefficient (r_d)	MSF	Ks	Cyclic Resistance Ratio (M=7.5)	Cyclic Resistance Ratio (M=6.66)	Cyclic Stress Ratio Induced by Design Earthquake	Factor of Safety	Comments
							(1)	(2)	(3)	(4)	(5)	(6)	(7)				(8)	(9)	(10)	(11)	(12)	(13)		
7	0	10	5	15	120		1.3	1.05	1.3	1.56	0.75	31.2	31.2	600	600	600	0.99	1.33	1.1	0.57	0.84	N/A	N/A	Above Water Table
9.5	10	12	11	15	120	18	1.3	1.05	1.23	1.20	0.75	22.6	26.7	1320	1258	1320	0.96	1.25	1.09	0.34	0.46	0.59	0.78	**Liquefiable**
14.5	12	17	14.5	30	120		1.3	1.05	1.3	1.05	0.85	47.6	47.6	1740	1459	1740	0.95	1.37	1.1	2.00	2.00	0.66	3.03	Non-Liquefiable
19.5	17	22	19.5	15	120	6	1.3	1.05	1.23	0.96	0.95	22.9	23.0	2340	1747	2340	0.92	1.19	1.03	0.25	0.30	0.72	0.42	**Liquefiable**
24.5	22	27	24.5	12	120	71	1.3	1.05	1.16	0.87	0.95	15.6	21.2	2940	2035	2940	0.89	1.17	1	0.22	0.26	0.75	0.34	**Liquefiable**
29.5	27	32	29.5	33	120		1.3	1.05	1.3	0.88	0.95	49.0	49.0	3540	2323	3540	0.86	1.37	0.97	2.00	2.00	0.77	2.61	Non-Liquefiable
34.5	32	37	34.5	56	120		1.3	1.05	1.3	0.98	1	97.7	97.7	4140	2611	4140	0.83	1.37	0.94	2.00	2.00	0.77	2.60	Non-Liquefiable
38.5	37	42	39.5	45	120		1.3	1.05	1.3	0.90	1	71.7	71.7	4740	2899	4740	0.80	1.37	0.9	2.00	2.00	0.76	2.62	Non-Liquefiable
44.5	42	47	44.5	70	120		1.3	1.05	1.3	1.12	1	139.0	139.0	5340	3187	5340	0.77	1.37	0.88	2.00	2.00	0.75	2.66	Non-Liquefiable
49.5	47	50	48.5	61	120		1.3	1.05	1.3	1.03	1	111.1	111.1	5820	3418	5820	0.74	1.37	0.86	2.00	2.00	0.74	2.70	Non-Liquefiable

Notes:

(1) Energy Correction for N_{90} of automatic hammer to standard N_{60}

(2) Borehole Diameter Correction (Skempton, 1986)

(3) Correction for split-spoon sampler with room for liners, but liners are absent, (Seed et al., 1984, 2001)

(4) Overburden Correction, Caluclated by Eq. 39 (Boulanger and Idriss, 2008)

(5) Rod Length Correction for Samples <10 m in depth

(6) N-value corrected for energy, borehole diameter, sampler with absent liners, rod length, and overburden

(7) N-value corrected for fines content per Eqs. 75 and 76 (Boulanger and Idriss, 2008)

(8) Stress Reduction Coefficient calculated by Eq. 22 (Boulanger and Idriss, 2008)

(9) Magnitude Scaling Factor calculated by Eqns. A.8 & A.10 (Boulanger and Idriss, 2014)

(10) Overburden Correction Factor calcuated by Eq. 54 (Boulanger and Idriss, 2008)

(11) Calcuated by Eq. 70 (Boulanger and Idriss, 2008)

(12) Calcuated by Eq. 72 (Boulanger and Idriss, 2008)

(13) Calcuated by Eq. 25 (Boulanger and Idriss, 2008)

LIQUEFACTION INDUCED SETTLEMENTS

Project Name	Proposed C/I Development
Project Location	El Monte, CA
Project Number	15G227
Engineer	DWN

Boring No. B-9

Sample Depth (ft)	Depth to Top of Layer (ft)	Depth to Bottom of Layer (ft)	Depth to Midpoint (ft)	$(N_1)_{60}$	DN for fines content	$(N_1)_{60\text{-}CS}$	Liquefaction Factor of Safety	Limiting Shear Strain γ_{min}	Parameter Fα	Maximum Shear Strain γ_{max}	Height of Layer		Vertical Reconsolidation Strain ε_v		Total Deformation of Layer (in)	Comments
				(1)	(2)	(3)	(4)	(5)	(6)	(7)			(8)			
7	0	10	5	31.2	0.0	31.2	N/A	0.04	-0.17	0.00	10.00		0.000		0.00	Above Water Table
9.5	10	12	11	22.6	4.1	26.7	0.78	0.07	0.13	0.06	2.00		0.013		0.31	Liquefiable
14.5	12	17	14.5	47.6	0.0	47.6	3.03	0.00	-1.39	0.00	5.00		0.000		0.00	Non-Liquefiable
19.5	17	22	19.5	22.9	0.0	23.0	0.42	0.11	0.35	0.11	5.00		0.000		0.00	Liquefiable
24.5	22	27	24.5	15.6	5.6	21.2	0.34	0.14	0.45	0.14	5.00		0.022		1.32	Liquefiable
29.5	27	32	29.5	49.0	0.0	49.0	2.61	0.00	-1.51	0.00	5.00		0.000		0.00	Non-Liquefiable
34.5	32	37	34.5	97.7	0.0	97.7	2.60	0.00	-5.85	0.00	5.00		0.000		0.00	Non-Liquefiable
38.5	37	42	39.5	71.7	0.0	71.7	2.62	0.00	-3.45	0.00	5.00		0.000		0.00	Non-Liquefiable
											Total Deformation (in)				1.63	

Notes:

(1) $(N_1)_{60}$ calculated previously for the individual layer

(2) Correction for fines content per Equation 76 (Boulanger and Idriss, 2008)

(3) Corrected $(N_1)_{60}$ for fines content

(4) Factor of Safety against Liquefaction, calculated previously for the individual layer

(5) Calcuated by Eq. 86 (Boulanger and Idriss, 2008)

(6) Calcuated by Eq. 89 (Boulanger and Idriss, 2008)

(7) Calcuated by Eqs. 90, 91, and 92 (Boulanger and Idriss, 2008)

(8) Voumetric Strain Induced in a Liquefiable Layer, Calcuated by Eq. 96 (Boulanger and Idriss, 2008)
(Strain N/A if Factor of Safety against Liquefaction > 1.3)

LIQUEFACTION EVALUATION

Project Name	Proposed C/I Development	MCE_G Design Acceleration	0.899 (g)
Project Location	El Monte, CA	Design Magnitude	6.66
Project Number	15G227	Historic High Depth to Groundwater	10 (ft)
Engineer	DWN	Depth to Groundwater at Time of Drilling	60 (ft)
		Borehole Diameter	6 (in)
Boring No.	B-17		

Sample Depth (ft)	Depth to Top of Layer (ft)	Depth to Bottom of Layer (ft)	Depth to Midpoint (ft)	Uncorrected SPT N-Value	Unit Weight of Soil (pcf)	Fines Content (%)	Energy Correction	C_B	C_S	C_N	Rod Length Correction	$(N_1)_{60}$	$(N_1)_{60CS}$	Overburden Stress (σ_o) (psf)	Eff. Overburden Stress (Hist. Water) (σ_o') (psf)	Eff. Overburden Stress (Curr. Water) (σ_o') (psf)	Stress Reduction Coefficient (r_d)	MSF	Ks	Cyclic Resistance Ratio (M=7.5)	Cyclic Resistance Ratio (M=6.66)	Cyclic Stress Ratio Induced by Design Earthquake	Factor of Safety	Comments
							(1)	(2)	(3)	(4)	(5)	(6)	(7)				(8)	(9)	(10)	(11)	(12)	(13)		
7	0	10	5	15	120		1.3	1.05	1.3	1.56	0.75	31.2	31.2	600	600	600	0.99	1.33	1.1	0.57	0.84	N/A	N/A	Above Water Table
9.5	10	12	11	16	120	4	1.3	1.05	1.25	1.21	0.75	24.7	24.7	1320	1258	1320	0.96	1.22	1.08	0.28	0.37	0.59	0.63	Non-Liquefiable
14.5	12	17	14.5	32	120		1.3	1.05	1.3	1.05	0.85	50.6	50.6	1740	1459	1740	0.95	1.37	1.1	2.00	2.00	0.66	3.03	Non-Liquefiable
19.5	17	22	19.5	48	120		1.3	1.05	1.3	0.99	0.95	80.1	80.1	2340	1747	2340	0.92	1.37	1.05	2.00	2.00	0.72	2.78	Non-Liquefiable
24.5	22	27	24.5	31	120		1.3	1.05	1.3	0.92	0.95	48.1	48.1	2940	2035	2940	0.89	1.37	1.01	2.00	2.00	0.75	2.66	Non-Liquefiable
29.5	27	32	29.5	60	120		1.3	1.05	1.3	0.99	0.95	100.4	100.4	3540	2323	3540	0.86	1.37	0.97	2.00	2.00	0.77	2.61	Non-Liquefiable
34.5	32	37	34.5	57	120		1.3	1.05	1.3	0.99	1	100.1	100.1	4140	2611	4140	0.83	1.37	0.94	2.00	2.00	0.77	2.60	Non-Liquefiable
38.5	37	42	39.5	46	120		1.3	1.05	1.3	0.90	1	73.9	73.9	4740	2899	4740	0.80	1.37	0.9	2.00	2.00	0.76	2.62	Non-Liquefiable
44.5	42	47	44.5	42	120		1.3	1.05	1.3	0.85	1	63.6	63.6	5340	3187	5340	0.77	1.37	0.88	2.00	2.00	0.75	2.66	Non-Liquefiable
49.5	47	50	48.5	39	120		1.3	1.05	1.3	0.81	1	56.0	56.0	5820	3418	5820	0.74	1.37	0.86	2.00	2.00	0.74	2.70	Non-Liquefiable

Notes:

(1) Energy Correction for N_{90} of automatic hammer to standard N_{60}

(2) Borehole Diameter Correction (Skempton, 1986)

(3) Correction for split-spoon sampler with room for liners, but liners are absent, (Seed et al., 1984, 2001)

(4) Overburden Correction, Caluclated by Eq. 39 (Boulanger and Idriss, 2008)

(5) Rod Length Correction for Samples <10 m in depth

(6) N-value corrected for energy, borehole diameter, sampler with absent liners, rod length, and overburden

(7) N-value corrected for fines content per Eqs. 75 and 76 (Boulanger and Idriss, 2008)

(8) Stress Reduction Coefficient calculated by Eq. 22 (Boulanger and Idriss, 2008)

(9) Magnitude Scaling Factor calculated by Eqns. A.8 & A.10 (Boulanger and Idriss, 2014)

(10) Overburden Correction Factor calcuated by Eq. 54 (Boulanger and Idriss, 2008)

(11) Calcuated by Eq. 70 (Boulanger and Idriss, 2008)

(12) Calcuated by Eq. 72 (Boulanger and Idriss, 2008)

(13) Calcuated by Eq. 25 (Boulanger and Idriss, 2008)

LIQUEFACTION INDUCED SETTLEMENTS

Project Name	Proposed C/I Development
Project Location	El Monte, CA
Project Number	15G227
Engineer	DWN

Boring No. B-17

Sample Depth (ft)	Depth to Top of Layer (ft)	Depth to Bottom of Layer (ft)	Depth to Midpoint (ft)	$(N_1)_{60}$	DN for fines content	$(N_1)_{60\text{-}CS}$	Liquefaction Factor of Safety	Limiting Shear Strain γ_{min}	Parameter Fα	Maximum Shear Strain γ_{max}	Height of Layer		Vertical Reconsolidation Strain ε_v		Total Deformation of Layer (in)	Comments
				(1)	(2)	(3)	(4)	(5)	(6)	(7)			(8)			
7	0	10	5	31.2	0.0	31.2	N/A	0.04	-0.17	0.00	10.00		0.000		0.00	Above Water Table
9.5	10	12	11	24.7	0.0	24.7	0.63	0.09	0.25	0.09	2.00		0.019		0.46	Non-Liquefiable
14.5	12	17	14.5	50.6	0.0	50.6	3.03	0.00	-1.63	0.00	5.00		0.000		0.00	Non-Liquefiable
19.5	17	22	19.5	80.1	0.0	80.1	2.78	0.00	-4.21	0.00	5.00		0.000		0.00	Non-Liquefiable
24.5	22	27	24.5	48.1	0.0	48.1	2.66	0.00	-1.44	0.00	5.00		0.000		0.00	Non-Liquefiable
29.5	27	32	29.5	100.4	0.0	100.4	2.61	0.00	-6.11	0.00	5.00		0.000		0.00	Non-Liquefiable
34.5	32	37	34.5	100.1	0.0	100.1	2.60	0.00	-6.08	0.00	5.00		0.000		0.00	Non-Liquefiable
38.5	37	42	39.5	73.9	0.0	73.9	2.62	0.00	-3.64	0.00	5.00		0.000		0.00	Non-Liquefiable
											Total Deformation (in)				0.46	

Notes:

(1) $(N_1)_{60}$ calculated previously for the individual layer

(2) Correction for fines content per Equation 76 (Boulanger and Idriss, 2008)

(3) Corrected $(N_1)_{60}$ for fines content

(4) Factor of Safety against Liquefaction, calculated previously for the individual layer

(5) Calcuated by Eq. 86 (Boulanger and Idriss, 2008)

(6) Calcuated by Eq. 89 (Boulanger and Idriss, 2008)

(7) Calcuated by Eqs. 90, 91, and 92 (Boulanger and Idriss, 2008)

(8) Voumetric Strain Induced in a Liquefiable Layer, Calcuated by Eq. 96 (Boulanger and Idriss, 2008)
(Strain N/A if Factor of Safety against Liquefaction > 1.3)